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THEORETICAL & EXPERIMENTAL INVESTIGATION INTO THE BASIC PROPERTIES OF BOULDER CLAY

FINAL TECHNICAL REPORT

by

R W KIRWAN principal investigator

T E GLYNN research assistant

SEPTEMBER, 1967

EUROPEAN RESEARCH OFFICE

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UNIVERSITY OF DUBLIN ENGINEERING SCHOOL

TRINITY COLLEGE DUBLIN

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PART 1
OF
FINAL TECHNICAL REPORT

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PREFACE

The work described in this report was performed under Contract No. DA-91-591-EUC4070-01-652-2284 entitled "Theoretical and Experimental Investigation into the Basic Properties of Boulder Clay" between the U.S. Army European Research Office and the University of Dublin, Engineering School, Trinity College. The research was conducted for the U.S. Army under research and development project number R & D 1286.

The sponsorship of the U.S. Army is gratefully acknowledged.

Soil samples, used in the experimental work, were supplied by the Electricity Supply Board of Ireland, Bechtel Corporation, Meath, Dublin and Leitrim County Councils.

ABSTRACT

The findings of a laboratory investigation into the response of compacted boulder clays in dynamic load tests are presented. Five soils, with liquid limits in the range 25 to 75, were subjected to pulsed deviator stresses applied in increments until a limiting permanent deformation was reached, or alternatively an axial stress of 18 p.s.i. was sustained without failure within 100,000 stress applications. Resilient moduli, covering a practical range of moisture contents and densities, were determined for each soil. The threshold stress, which separates the regimes of mainly elastic from the inelastic behaviour was established by an analytical procedure. The application of the threshold stress concept to flexible pavement performance is discussed.

The compaction characteristics and the C.B.R. vs moisture content and density relationships were determined, and were correlated where practicable with the results of the repeated load tests.

Conventional shear tests, slow undrained triaxial and unconfined compression, were performed. These test results are reported in abbreviated form.

The repeated load test equipment was fabricated within the contract period. The samples were tested under repeated loading in undrained triaxial compression. Comparatively large samples were used, for which development of some special pieces of apparatus was necessary. A new system for measuring lateral deformation of samples under load was devised.

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Chapter 1

INTRODUCTION

Boulder clay, otherwise known as glacial till, is a deposit of unstratified clay or sandy clay containing subangular stones of various sizes scattered irregularly throughout its mass; the stones are not necessarily all of boulder size. The composition varies from high clay contents (over 50%), to that just sufficient to impart the cohesive characteristics of boulder clay; the average clay content is 20 per cent. Boulder clays have a widespread areal distribution in lowland regions of the British Isles, and occur in strata varying in thickness from a few to several hundred feet. The Soil Mechanics Department at Trinity College has had a long-standing interest in the properties of boulder clay. This interest stems from an awareness of the problems encountered in building construction and road works on glacial deposits in Ireland. Previous work in the Department concentrated on compressibility and shear strength determinations using static tests. (Kirwan and Daniels, 1961)* The award of the present contract has made it possible to engage in the considerably more complex field of dynamic testing. Part I of this report presents the results of both dynamic and static tests performed on selected boulder clays but the main emphasis is on dynamic testing.

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*References are listed at back in the order in which they first appear in report

Dynamic testing is a many faceted study. It covers the response of soil samples to pulse loadings of the magnitudes, periods and frequencies associated with such a diversity of force-function generators as seismic tremors, blasting, rocket launching, vibrating machinery, shipthrust, wind gusts, ocean waves and moving traffic on railways, and on road and airfield pavements. The response of soils subjected to dynamic loading is complex, for, in addition to such factors as rate of loading and pore pressures that influence static test results, there are inertial and damping phenomena inherent in most dynamic test procedures. Clayey soils are particularly difficult because they exhibit elastoplastic behaviour* departing markedly from a linear load-deformation response in virtually all laboratory and field tests.

Three dynamic techniques are currently employed to arrive at response characteristics:-

(a) tests to measure velocity of stress wave propagation, from which stiffness moduli are computed using the theory of elasticity.

(b) vibration resonance measurements which again depend on elastic theory to obtain moduli indirectly.

(c) the direct dynamic tests which consist of applying either stresses or deformations of preselected magnitude to the soil specimen and observing the response.

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* Definitions of uncommon terms are given in Glossary at back of report.

No single test has been devised that yields a set of parameters that have a general application to the solution of the manifold problems encountered in dynamically loaded foundations Records of investigations show that stiffness moduli, computed in a manner analogous to Young's modulus, decrease with increasing major principal stress (or deviator stress). Clays in particular, show a nonlinear relationship between stiffness modulus and principal stress level, as well as the decrease in stiffness modulus with increasing stress. Moduli determined from wave propagation measurements show the greatest numerical values, vibration resonance methods yield next highest stiffness moduli, and the direct loading tests, either transient or repeated loading, yield the lowest values. This is a result of the fact that the wave method applies the lowest stress and the direct method applies the highest stress. The stress level accounts largely for the broad range of values of the dynamic constants reported for even similar soils. (Vesic and Domaschuk, 1964, Wilson and Dietrich, 1960). Hence care must be exercised in comparing moduli from different sources, and in selecting moduli from listed values, to ensure that the figures quoted are in fact applicable to the specific problem.

The non-recoverable component of deformation comprises an increasingly large proportion of the total deformation if the major principal stress is raised in any of the above mentioned tests. Even at stress levels well below those required for general yielding of a soil, plastic deformation will accumulate with the number of stress applications. Creep and stress relaxation are

most pronounced in clayey soils.

Evidently for results of laboratory tests to be directly applicable to a specific dynamic problem, the method of testing should simulate as closely as possible the stress level, duration of stress application, waveform and frequency of stresses anticipated in the full scale application. Repeated load tests, where an 'at rest' interval is provided between successive stress applications, most closely simulates the prevailing conditions in soil masses supporting travelling loads on pavements.

The primary purpose of the investigation, described in this report, is to determine the effects of repeated loading on samples of compacted boulder clays tested under conditions that simulate the stress levels and waveforms obtained in field measurements on subgrades beneath flexible pavements reported elsewhere. (Wiffin, 1955). The secondary purpose is to examine the results of repeated load tests, together with conventional tests, to see what correlation, if any, exists between the results of dynamic and static tests.

Although the methods developed are primarily intended for testing soils which are used as subgrade materials for roads and airfields, the system can be readily extended to embrace the testing problems associated with soil subjected to lateral loading from piles which absorb the thrust of a ship berthing. Also the methods could be adopted to meet the problems of soil behaviour encountered in railway engineering.

Related work on repeated load testing has previously been reported by a number of authors, principally Seed et al., (1955, 1957, 1958, 1962), Johnson and Yoder, (1962), Larew and Leonards (1962), Kawakami and Ogawa (1963), Grainger and Lister (1962). Between them these researchers have investigated a wide range of the variables that influence the results of repeated load tests. However, the records of their work available to us show that the majority of repeated load tests were performed on samples 2-inch diameter and smaller in triaxial compression tests. Most authors reported limitations on the stress waveforms imposed by their equipment.

The apparatus developed by us appears to give greater flexibility in the choice of stress waveforms than those produced by equipment used in the research referred to in the above reports. Furthermore, we have carried out the repeated load tests on 4-inch diameter samples which permits the use of 1.5-inch maximum particle size if needed. The programme of conventional testing run concurrently with dynamic tests is thought to offer certain advantages when it comes to a critical examination of existing design procedures.

Part I of this report is a record of the experimental work and an analysis of the data. Part II is devoted to theoretical work, where an attempt is made to utilize both the results of the experimental work and data reported by others. The theoretical work may be treated as a separate contribution; the scope of which was not specifically outlined in the contract.

1.1. Scope of Investigation.

Design, assembly and proving of equipment to test two 4-inch diameter specimens in repeated loading triaxial compression incurred spending six months of the one year contract on this aspect alone, rather more than we had originally estimated. Consequently within the time available for dynamic testing, a limited number of samples were sought that would be representative of the range of glacial clayey tills found in Ireland. It was felt that plasticity may well prove to be the property underlying the behaviour of these fine grained soils in repetitive loading. The Atterberg limits of a dozen boulder clays were determined, and from these five soils were selected with liquid limits within the range 25-75, with at least one soil located in each of the plasticity zones in the Casagrande plasticity chart.

The particle size distributions of the five clays were adjusted to fit an average grading curve obtained by inspection of a large number of natural gradations. The grading control was confined to the coarse fraction and fixed the percentage retained on the 3/16" British Standard sieve (U.S. sieve No. 4.). The artificially graded samples were used in all tests - specimen size permitting. It would have been of interest to include a sample with liquid limit over 70 as such clays occur occasionally in glacial deposits; however time was not available to accomplish this.

We extended the scope of the testing outlined in our contract proposal to include results of conventional triaxial shear and unconfined compression tests, for reasons

explained later. The results reported herein consist of the following:

Complete moisture density relationships obtained in compaction tests on the five boulder clays.

Dynamic moduli and permanent deformations for not more than five specimens of any one soil; a reasonable spread in moisture contents and densities was attained in the specimens tested from four soils.

Unconfined compressions on specimens that had previously been tested in repeated loading.

California Bearing ratios over a reasonable range of moisture contents for four soils.

Slow triaxial shear tests on 1.5-inch diameter specimens with volume change measurements.

A short feasibility study on the hitherto infrequently attempted objectives of measuring pore-pressure, and measuring lateral deformation of the specimen in repeated load tests is presented. An experiment was devised to demonstrate that electro-osmosis can be employed to increase the degree of saturation from that of the 'as compacted' state. The method is intended to improve control over soaking of samples compacted dry of optimum moisture content. Flocculated structure is thus more readily preserved in soils of low permeability. Ingress of water is virtually instantaneous with application of electric potential.

Impact compaction was employed in preparation of specimens. Consequently the soil structures obtained were most probably somewhere between the structures associated with kneading and with static compaction methods, i.e., kneading compaction tends to impart the most disperse structure to compacted clay. (Seed 1961).

Chapter 2

GENERAL DESCRIPTION OF MATERIALS AND TESTING PROCEDURES

2.1. Soils Used in Investigation

Irish boulder clays consist of an assortment of detritus deposited in the Pleistocene and earlier glaciations. Generally, every particle size is present from the colloidal to stones of boulder size. The mineralogical composition of the parent material (rocks) appears to be a significant factor - imparting to the soil its existing texture and colour. Argillaceous and calcareous rock formations were eroded to produce dark fine grained soils. More resistant siliceous rocks broke down into light coloured coarse grained soils. Notwithstanding the effects of weathering, the parent material is traceable in the fine grained soils that occur in the central lowlands. On the other hand, the igneous rocks in the mountainous coastal belt are the parent materials of the light coloured predominantly silt and sandy-silt tills found there.

Generally, boulder clays are highly preconsolidated, in the Dublin area the overconsolidation pressure is estimated at 30 tons per square foot. Despite the high over-consolidation, boulder clays are characteristically of low sensitivity; sensitivities of two or three are common. Peat accumulated over the clays where the postglacial topography favoured swamps Exposed boulder clays are found mostly in drumlins (whaleback hills or ridges). The thickness of the weathered

zone is quite irregular and appears to be linked with the permeability of the original deposit. The C-horizon occurs between 2-20 feet below ground level. Weathered boulder clay is normally a lighter colour than the underlying material. Weathering increases compressibility and lowers the shear strength. Road base courses more often than not rest on the weathered zone.

The geographical locations of the soils used in the investigation are shown in Figure 1.

Arigna Boulder Clay is a black medium plastic inorganic soil, tough in the undisturbed state, sticky when remoulded in the presence of additional moisture. The liquid limit equals 40 and plasticity index is 22. The sample used in this investigation was taken from the bottom of a 40 foot deep excavation for a new road. The excavation was in the trailing slope of a drumlin which had a plan area of two square miles approximately. The sample represents unweathered material, and the stratum was quite uniform over the entire face and the bottom of the excavation.

The job afforded a good opportunity to observe the problems encountered in the excavation of, and road construction on certain boulder clays. A medium power excavator (19 R.B.) was having difficulty excavating the tough soil, using a face shovel. Yet, when exposed to a few rain showers, softening to viscous fluid consistency occured in the upper eighteen inches. A stretch of the new road was a problem due to intrusion of softened subgrade soil into a crushed stone base course. Passing trucks produced noticeable pumping of slurry

from the unsurfaced base course. A remedial measure under trial consisted of a three inch layer of flyash placed directly on the subgrade; the flyash was obtained from a nearby coal burning electric generating station. The flyash barrier appeared to be a promising solution, as one could readily distinguish between the two sections by noting the absence of clay staining on top of the base course laid on the treated subgrade. The thickness of the base course on the treated stretch was 1'-6" while on the untreated section a base course 1'-9" thick had been laid. The California Bearing Ratio determined on an undisturbed sample equalled 5% at 15.3 per cent moisture. The Arigna boulder clay covers an area of over 400 square miles, and is remarkable for poor drainage properties.

Dublin Boulder Clay was obtained from a shallow road excavation close to Dublin Airport. It is a brown medium plastic inorganic material. The liquid limit is 32, plasticity index 15. The sample used in this investigation was taken from the weathered zone at about 4 feet below the surface. The soil has a low sensitivity of the order of 2.

The Dublin boulder clay is regarded as a competent foundation material, and softening on exposure is not normally a serious problem. Slopes at the Airport site were cut at about 1.5 to 1 and were quite stable and firm on the face. This is the most common type of boulder clay, and covers several thousand square miles in the Central Plain. At depth (over 20 feet) the dark brown clay changes to a black boulder clay of similar grading and plastic indices but very much higher

strength and lower compressibility. In foundation work the bearing capacity is taken at 2 T.S.F. for the dark brown and 4-8 T.S.F. for the black clay. Agreement is not general as to how the colour difference emerged, the majority of geologists maintain that the difference is due to deposition by separate glaciations.

<u>Erne Boulder Clay</u> is a dark brown silty clay of low plasticity. The liquid limit varies between 22-35, and the plasticity index is about 15. The sample was obtained from a borrow pit that had been used previously for impervious core material in low embankments on the Erne Hydro-electric scheme. The soil has a decidedly silty appearance and the clay fraction appears to possess low cohesion. This soil was located after a fairly extensive search for a boulder clay of low plasticity. The sample may have been slightly weathered.

<u>Gortdrum Boulder Clay</u> is a red inorganic clay. The liquid limit is 49, plasticity index 25. This is on the borderline between the medium and high plasticity. The sample used in the investigation was supplied by Bechtel Corporation's contractors on mining operations at Gortdrum. The clay was taken from a borrow pit used for grading the ore yards. The extent of the deposit has not been investigated, but it is fairly characteristic of boulder clay deposits in the south west of the country, where the socalled Old and New Red Sandstone rock formations occur.

<u>Slane Boulder Clay</u> is one about which there is doubt concerning its origin. In the natural state it is very fine grained with only occasional stones and a high organic content. However, the sample was obtained from

a drumlin at an elevation relatively high in flat countryside. It appears to be a late glacial deposit, if indeed a glacial deposit at all.

Our attention was drawn to this soil by failure of a two mile stretch of existing road. The road had carried light traffic and occasional trucks over the years, it required only occasional repairs to the surfacing.

A few years ago a quarry was opened and the road became a truck route. Failure of the road occured in a matter of a few months. Very severe rutting developed on the laden truck lane, and less pronounced ruts occured on the opposite lane of the two lane road. The remedial measure employed was to excavate the clay subgrade alongside the roads and widen it by placing up to 5 feet of base course of crushed stone in longitudinal trenches and resurfacing. The sample used in this investigation was taken from the spoil tip of material taken from the trenches. The liquid limit depends on the organic content, and varies from 51 for the inorganic soil to 75 for the organic soil.

The classification tests are shown in Figures 2 and 3. The mineralogical analysis of the soils is given in Appendix I.

2.2. Preparation of Samples For Tests.

The boulder clay was broken into small lumps and left to air dry in large trays. When the soil was sufficiently dry, these lumps which were quite hard were broken down using a wooden mallet -taking care not

to crush the grains. The pulverised soil was sieved through a 3/16" B.S. sieve. The composite samples were prepared by adding 10 parts 3/16" - 3/8" size, 10 parts 3/8" to 3/4" to 80 parts of minus 3/16" by weight. The minus 3/16" possessed about 2 per cent moisture, but the final grading was in all cases close to 20 per cent retained on 3/16" sieve. The volume of water required to bring the soil to the desired moisture content was calculated. The soil and water were then mixed thoroughly in a mixing machine; water and soil being added intermittently so that the water, which was sprayed from an atomizer was distributed evenly throughout the soil. The wet soil was packed into tins and allowed to mature.

Compaction test results are shown in Figures 5 and 6. For each moisture content the same specimen was compacted at three different compactive efforts:- Standard A.A.S.H.O., Modified A.A.S.H.O., and an intermediate consisting of 55 blows per layer on three layers using a 5.5 pound rammer, with a drop of 12 inches in the standard proctor mould. Because the soil was re-used to obtain a set of three densities, we checked for particle crushing by wet sieving before and after the three compactive efforts were applied.

Typical results of the wet sieving are as follows:-

SIEVE SIZE	PER CENT PASSING SIEVE BY WEIGHT					
	Initial Grading	After compaction at moisture contents				
		8%	10%	12%	14%	16%
3/4"	100	100	100	100	100	100
3/8"	90	91	90	92	90	89.6
3/16"	81	81.5	82	83.5	82	83
		10%	12%	14%	16%	
3/4"	100	100	100	100	100	
3/8"	90	92	90	90	90.4	
3/16"	81	84	81	82	80.8	

The gradings were obtained on the Arigna soil which had the weakest coarse particles i.e. shale. We conclude that particle crushing was not significant.

The impact compaction machine is shown in Figure 4.

2.3 Strength Tests

The results of strength tests reported herein consist of C.B.R., unconfined compression and slow triaxial shear.

The C.B.R. tests are a requirement of the contract. The tests are required for correlating the C.B.R. method of pavement design with the elastoplastic approach, which considers resilience and plastic deformation of the subgrades. The relationships between C.B.R./density/moisture content were investigated for Arigna, Dublin, Gortdrum and Slane samples. The C.B.R. tests were performed in accordance with British Standard 1377. The requirements of the standard are the same as the procedure for C.B.R. tests described in "Principles of Pavement Design," Yoder 1959, pp. 204-210. The only

essential difference concerns soaking the specimen under a surcharge equal to the overburden pressure. The Road Research Laboratory considers soaking to be too extreme. (Road Note No. 29 pp. 27). In any event, as we are concerned at this stage with investigation of trends in results rather than design criteria the question of soaked vs. unsoaked samples is not of vital importance, in our opinion.

The C.B.R. equipment is shown in Figure 4. Recall, that the C.B.R. tests were on the artificially graded soils shown in Figures 2 and 3. The results of the C.B.R. tests are plotted vs. density and moisture content in Figures 7,8,9,10 and 11. The C.B.R. values of the Erne soil were not fully investigated.

2.4 Slow Triaxial Shear Tests.

A question that arises in theoretical work is the degree of mobilization of the cohesion and internal friction parameters by strain. To establish the mobilization relationships, we performed undrained strain-controlled triaxial tests on 1.5-inch diam. x 3-inch specimens of the minus 3/16" matrix. The tests were run on Arigna and Dublin boulder clays. These tests involved volume change measurements from which the mobilization of internal friction can be inferred from plastic theory following the work of Druker and Prager (1952).

The specimens were prepared by compacting the soil in the standard proctor mould and varying the compactive efforts. When the soil was compacted it was jacked out of the mould against the cutting edge of three 1.5-inch diameter thin walled steel tubes - symmetrically placed

and rigidly fixed in position. The specimens were allowed to remain in the tubes for two days at constant moisture content. Then they were removed from the tubes and left for a further five days in airtight jars. The samples were tested undrained at an axial strain rate of 0.2% per min. The volume change in the specimen was measured by the commercial equipment shown in Figure 12. Typical results of the slow undrained triaxial tests are shown in Figures 13, 14, and 15.

2.5 Unconfined Compression Tests.

Unconfined compression tests were performed on the specimens that had previously been subjected to repeated load tests. The tests are intended to provide information on strain hardening produced by repeated loading and also data on stresses at failure.

The unconfined compression tests were performed in a conventional strain controlled triaxial machine at axial strain rate of 0.75% per min.

The results of the unconfined compression tests are tabulated below:-

TABLE I

Results of Unconfined Compression Tests on Samples Tested earlier in Repeated Loading.

(See page 18.)

TABLE I

Results of Unconfined Compression Tests on Samples Tested earlier in Repeated Loading.

Sample Number	Moisture Content (per cent)	Axial Stress in p.s.i. (@ 5% strain)
A 7	11.4	53.5
E 1	12.0	23.0
E 2	9.9	48.6
S 1	17.4	51.7
S 2	20.0	18.8
S 3	23.4	30.2
G 1	13.4	44.9
G 2	14.4	44.2
G 3	15.6	16.4

2.6. Dynamic Tests

As stated earlier, the dynamic test that most closely simulates the state of stress imposed by moving wheel loads on pavement components is the so called repeated load test. It is particularly suited to testing subgrade materials. In this section, the equipment and procedure for performing repeated load tests on cohesive soils are described.

The repeated load testing machine used is a development of the prototype constructed in the British Road Research Laboratory* and reported in Geotechnique (Grainger and Lister 1962). The machine is of the 'controlled stress' type. The specimen is tested in triaxial com-

*Road Research Laboratory will be abbreviated to R.R.L. henceforth in this report.

pression with the axial and lateral stresses applied as in-phase pulses whose waveforms and periodic times can be varied to simulate the stress conditions that have been measured in the field, and in full scale experiments on pavement structures The desired waveform of stress is produced mechanically Pulse durations and frequencies to simulate traffic speeds up to 30 m.p h are attainable with the machine.

The R.R.L. design was adopted after a review of several possible systems, some of which had been used for repeated load testing. It was considered that hydraulic and compressed air systems would necessitate a complicated arrangement of valves, and valve operations. The hydraulic and compressed air systems examined placed undesirable restrictions on the waveforms of axial and lateral stresses (Seed and Fead 1959, Yoder and Haynes 1963). Electromagnetic pulse assemblies were deemed unsuitable as these are known to possess non-linear load displacement outputs unless elaborate measures are devised. However, the three methods mentioned above have an advantage not readily achieved in the R.R.L. system; shorter duration pulse and shorter 'at rest' intervals can be obtained, so it appears that higher traffic speeds (up to 100 m.p.h.) can be simulated Very high frequency pulses can of course be obtained in an electromagnetic system; but in practice the greatest distress in the pavement is caused by slow moving traffic - other things being equal.

The R.R.L. design had to be modified considerably to cope with the larger size samples used The modifications were not merely a scale up of the

original model, but involved development of components not required in testing smaller specimens. In fact, a completely new layout was undertaken using the principles proven in the prototype and the experience in its use. The research graduate visited the R.R.L. in order to be briefed on developments there since the original work was published. They had duplicated the original loading frame to test two samples simultaneously, and were also investigating special features of the equipment such as piston friction in the triaxial cells.

2.7. Description of Apparatus for Repeated Load Testing.

For descriptive purposes it is convenient to consider the apparatus as made up of three assemblies:-

(i) A loading frame, consisting of a system of levers, springs, balancing components, all operated by cams driven by an electric motor through reduction gears.

(ii) A triaxial press which incorporates the cell, a deformation compensator and a hydraulic load multiplier, the latter to increase the thrust developed by the loading frame.

(iii) Mechanical and electronic instruments for measuring stresses, deformations, and pore water pressure.

A block diagram of the equipment is shown in Figure 16

The loading frame is mounted on a vibration isolated

concrete foundation. It accomodates the pulsing system for two triaxial cells. The tests can be run singly or in pairs; the stresses can be varied independently on each sample. A diagrammatic layout of the apparatus to show the method of operation is shown in Figure 17. Figure 18 is a photograph of the complete installation. A half horse power three phase electric motor M (960 r.p.m.) drives the cam wheel shaft via a continuously variable gearbox. The speed of the shaft can be varied between 3 and 21 r.p.m. A spider type coupling is installed between motor and gearbox. The shaft carries two main cam wheels, one to each triaxial cell for producing the pulsating stresses.

Axial Stress.

The axial loading system for a single cell operates as follows, and is merely duplicated for more than one triaxial cell.

The cam wheel A can carry one or more cams bolted to its periphery. It runs in continuous contact with a cam-following wheel on the main lever which is pivoted on an inset ball race at B. The opposite end of the main lever has a groove machined out fully, in which slides a floating pivot consisting of a roller bearing to which is attached a link rod to the upper of two plates housing the compression springs C. As the main lever is tilted by the cam, the springs are compressed and the load is transferred via the lower plate and stirrup hanger to the loading lever D, which is supported on an inset ball race at E. The position of the stirrup hanger by which the force is applied to the load lever can be varied along the lever. A single

roller bearing F rides on the top edge of the load lever, and provides the pivot which can be locked at the desired leverage, i.e. the position of the pivot on the upper lever can be locked, whereas, the pivot in the cam driven lever is free to slide and take up its equilibrium position in the machined slot. Adjustment of the length of stirrup hanger is made in the compression spring housing. The load lever is counterbalanced by the tension spring G. In the R.R.L. machine knife edge pivots were used, and although knife edges give more precise settings, clamping would appear to be more difficult. The load is in turn transferred to the specimen in the triaxial cell H via the hydraulic load-multiplier K, proving ring L and compensating device T described later. By using cams of different shapes, the waveform and duration of pulse can be varied. Figure 19 and 63 shows the waveform of axial stress.

Cell Pressure.

The lateral pressure on the sample is applied as a pulse in phase with the axial stress and it also is produced by a cam mechanism. However, a single cam produces the pressure pulse for the two cells. The cam is mounted on the side of the cam wheel. By means of a cam follower and rocker arm N a horizontal thrust is obta obtained which drives a piston pump. The compression generated by the piston pump is transmitted to the water surrounding the sample in the triaxial cell. The pressure can be varied by altering the volume of trapped air in a reservoir R on the line between pump and cell. The water pressure is directed to the appropriate cell by a pair of solenoid valves S, actuated by micro switches and another small cam wheel mounted on the main drive shaft. The solenoid valves are set to open in advance of the axial pressure pulse thus ensuring that the sample is confined.

Figure 65 shows the waveform of cell pressure.

Triaxial Cells.

The triaxial cells are mounted in separate presses, each press consisting of a base plate and upright frame resting on a concrete pedestal almost one ton weight. Commercial triaxial cells are used. The only modification required in a cell designed for static testing is the honing down of the piston to obtain a free falling fit. In fact one early test using a new cell had to be abandoned due to piston jamming.

The press supports the hydraulic load-multiplier, the proving ring, and compensating device, so practically no load is imposed on the sample during 'at rest' interval. The load is transferred to the metal cap above the specimen via a hemispherical seating mating with the loading piston. The triaxial press is shown in Figure 20.

It was found that the original R.R.L. design would not work satisfactorily if high loads were demanded of the cam mechanism. When the stiff springs, necessary to develop high load, were installed in the compression spring housing, the result was severe back-lash as the cam passed the position of maximum displacement. The jump in the cam caused distortion in the waveform corresponding to the unloading condition.

The hydraulic load-multiplier was devised as a simple means of obtaining a smooth cam action at all values of axial load. The device consists of a double section cylindrical capsule with a brass plunger

operating in a port in the upper (input) end; the opposite enlarged end is sealed with a rubber diaphragm, that, in turn, bears on a flat circular piston head. The piston is rigidly connected to the top boss of the proving ring. The hydraulic load-multiplier is shown in Figure 20. A brand of non-mineral hydraulic fluid (Shell Super H) is used for filling the capsule to the exclusion of all air bubbles. The pulse from the loading frame is transmitted by oil pressure developed by the reciprocating plunger. It was not readily feasible to prevent oil leakage by the sides of the plunger, and at the same time preserve its free moving action. To overcome leakage a constant back pressure is maintained in the oil by a pressurised sump that replenishes losses. The return of the plunger from a depressed position is further assisted by spring loading its stem. Magnification of the load lever thrust is close to the theoretical. Attenuation of the stress pulse is minimal, and, in fact, whatever attenuation does occur is advantageous, supressing unwanted high frequency oscillations originating in the compression springs C.

The piston-proving ring assembly is suspended on springs attached to the triaxial press. The oil pressure in the capsule is set to counterbalance exactly the weight of the assembly. Thus no force is exerted on the sample when the plunger is not in the depressed position. The compensating device shown in Figure 20 maintains the contact between the proving ring and the piston of the triaxial cell at all times. With normal elastic deflection of the specimen, the piston of the triaxial cell is in contact with the compensating device via the steel ball between it and the piston. As deformation develops

in the specimen, the piston and the steel ball lose contact with the lower end of the compensator sleeve. Immediately this happens the compensator unit rotates under the action of two small weights, suspended from the cord wound around the barrel of the compensator, causing it to move down the threaded studding until contact is again made.

Permanent deformation, if not compensated would modify the form of subsequent loading cycles and introduce some measure of impact. This effective device is a feature of the R.R.L. machine and was adopted with very little modification. The number of stress applications is registered on a mechanical counter actuated by a cam on the main drive shaft of the loading frame.

Measurement of the axial load, cell and pore pressures, axial and lateral deformations is made at regular intervals throughout the test.

<u>Axial Load Measurement</u>.

The deflexion of the proving ring is measured by means of a displacement transducer. The transducer is of the inductance type and gives virtually a linear output over it's rated range. A dial gauge mounted in parallel with the transducer can be coupled, when required, to check visually the load being applied or for calibration. The deflexion of the sample, and the movement of the proving ring, is small and introduce no significant inertial effects.

Cell and Pore Pressure Measurements.

Cell pressure is measured with an inductance type pressure transducer The transducer is a differential model permitting both pore and cell pressure to be measured simultaneously when the sample is tested in the undrained condition. No-flow valves are fitted on the hydraulic lines to the transducer - a Bishop type piston valve on the pore pressure port and a Klinger valve on the cell pressure inlet A Bourdon test gauge is fitted in the hydraulic lines for calibration and checking the transducer

Axial Deformation Measurement

The resilient axial deformation in the sample is measured with a displacement transducer The active element of the transducer follows the movement of the piston of the triaxial cell. An arm clamped to the piston of the triaxial cell carries the active element which can be adjusted with a micrometer The micrometer is a permanent fixture and permits direct calibration at any time during the test The output of the transducer is linear over displacements up to 1/4-inch for the models employed. Indirect calibration by substituting a known inductance in the measuring bridge is an alternative, and was in fact the method adopted at the R.R.L. The permanent axial deformation is measured with a micrometer graduated in 0.001 inch divisions. This micrometer can be lowered on to the above mentioned arm at any time during the test; the contact is indicated by a pilot light The micrometer and the transducer are fixed in a rigidly supported cage W mounted clear of the triaxial press as shown in Figure 17.

Lateral Deformation Measurement.

The resilient lateral deformation of the sample is measured by another inductance type transducer in - stalled in the specimen as shown in Figure 57. No entirely satisfactory commercial transducer could be procured for this purpose. With some improvisation, an instrument suitable for measuring resilent lateral deformation was assembled. At the present state in the development, the transducer is not capable of measuring permanent lateral deformation due to drift of the zero setting caused by self-induced temperature fluctuations. In the future the transducer will be mounted outside the rubber membranes. Figure 19 shows the waveform obtained from the lateral deformation transducer.

Electronic Recording.

The displacement and pressure transducers feed into the same electrical circuits. Suffice it to say here, that the transducer is connected to circuit elements in an amplifier demodulator to form a half bridge network. This network is fed from a carrier source, e.g. 3 kcs, and is adjusted for balance so that, as long as the transducer element is not actuated, no output signal is produced. The parameter being measured actuates the transducer element and unbalances the electrical bridge so that the latter produces an output signal in the form of a suppressed carrier, amplitude modulated to an extent proportional to the instantaneous change in inductance of the transducer. The modulated signal is amplified and then passed to

a phase-sensitive demodulator, fed from the same carrier source, where it is converted into a rectified but un-smoothed signal of twice the carrier frequency, the polarity depending upon the direction of the actuating parameter. The signal is finally smoothed in a low-pass filter network to provide a polarised output current, or voltage, proportional to the original movement of the transducer element. The current drives galvanometers in an U.V. recorder. Incidentally, the electronic system is also compatible to resistance strain gauge measurements and transducers based on strain gauges, but according to the manufacturers it is not suitable for use with capacitance type transducers.

Two low gain and one high gain amplifier demodulators were procured. The U.V. recorder has six channels, four of which are used. Pertinent data from the manufacturers (S. E. Laboratories Feltham England) are:

Transducers	Accuracy	Stability
Displacement type:	0.5%	0.1% of F.S.D.
Pressure type:	0.25%	0.5% of F.S.D.
Operating Temperature:	-20°C + 80C	

Galvanometers

Natural frequency 450

Flat frequency response $\pm$ 0.5% c/s

Amplifiers: Linearity better than 0.25%
Stability over temperature range
within $\pm$ 0.02% /°C.

2.8 Performance of Repeated Load Test Apparatus.

The performance of the apparatus for tests on 4-inch diameter by 8-inch specimens is as follows:-

Axial stress range 1 to 18 p.s.i. $\pm$ 0.25 p.s.i.

Cell pressure 0.5 to 8 p.s.i. $\pm$ 0.25 p.s.i.

Deformation measurements:

permanent sets down to 0.001"
resilient to 0.0002"

Number of repetitions:

no restriction

Frequency:

up to 20 per minute, optimum working at 10 repetitions per minute.

Pulse duration:

One tenth of time interval between pulses.

The only drawback of any consequence, found in operation of the machine to date, is the harmonics on the cell pressure waveform which are caused by the action of the solenoid valves. Piston pumps working off separate drives would eliminate the problem as the solenoid valves could then be omitted.

2.9. Test Procedures.

The soil was prepared by air drying as described earlier. The samples were compacted in a split-mould

using six layers of soil at the preselected moisture content. The compactive effort per layer was gauged to the desired density and every effort was made to obtain uniformity. The compacted specimen was placed between perspex (lucite) cap and base, and surrounded by two rubber membranes with a film of non-mineral oil between the membranes, the membranes were sealed against the cap and base by means of neoprene O-rings. The specimens were stored for periods up to fourteen days to permit some measure of thixotropic strength regain, and to equalise the soil moisture. Direct from storage the entire specimen was placed under water in the triaxial cell, in the usual manner The O-ring seals were now placed against the metal cap and base of the cell Several repetitions of cell pressure only were applied for seating the end bearing surfaces of the sample. The loading consisted of an initial low stress followed by further increments of axial stress applied in sequence to the same specimen. Each load increment was maintained until the permanent deformation became negligible, and there was no significant change in the resilient modulus with a number of stress repetitions. The specimen was tested until the total permanent deformation was of the order of 0.2 ins. The greatest number of load repetitions applied at any one stress is 100,000.

The resilient deformation was converted to a strain by dividing it by the original height of the sample, and then using the stress read for that value of the strain to compute the resilient modulus. Fluctuation in the deviator stress of the order of 1% were thereby accounted for in moduli determinations.

Two corrections to the deviator stress were examined and considered negligible i.e. membrane restraint correction and the effect of the pulsed cell pressure on the zero setting of the proving ring. The membrane correction is not readily ascertained at the small strains imposed, but conventional triaxial testing would indicate no serious discrepancy in it's omission even at failure strains. The effect of pulsing the cell pressure calls for a correction that reduces the apparent deviator stress by the quantity $\sigma_3 a/A$ where σ_3 equals cell pressure, a is the area of the triaxial piston and is the area of the specimen cap. For our triaxial cell the correction is 3% of the cell pressure, or less than 2% of the deviator stress.

The axial deformation vs number of stress applications is shown on the odd numbered Figures 21 through 55. The resilient modulus vs number of stress applications is shown on the even numbered Figures 22 through 56.

Measurements of pore pressure and resilient lateral deformation were performed merely as a feasibility study; no systematic records of either were attempted. The study showed that the resilient lateral deformation can be obtained at any time during a test using the method of installing the displacement transducer shown in Figure 57. Measurement of pore-fluid pressure presents all the problems found in static tests, such as differences in pore air and pore-water pressures, de-airing of equipment and time lag in the response of the measuring system. The differential pressure transducer appears satisfactory, but further work is needed to ensure reliable pore pressure readings. The method of pore pressure measurements using the inductance transducer is shown in Figure 57.

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Chapter 3

ANALYSIS OF TEST RESULTS

In this section trends in the data obtained for the five soils will be examined to see what generalities can be formulated. The diversified scope of the programme with only a limited number of any one kind of test lends itself to broad deductions, rather than detailed analysis of the influence of particular variables. Occasional liberties with the data will be taken to seek out the relationships of interest, notwithstanding that further experimental work would be necessary to verify the deductions in some instances. At this juncture, it is perhaps worth noting that comparatively few data obtained from repeated load testing are reported in the soil mechanics literature. Apart from the publications of Seed and coworkers at the University of California, there seems to be a paucity of systematic data on the repeated load topic.

The test results will be dealt with in the order in which the experimental work was described in the previous section. Plots used in the analysis will show the measured parameters, where available, with the qualitative interpretations superposed.

3.1. Relationship between C.B.R., moisture content and density

The C.B.R. values are plotted against dry density and moisture content in Figure 7 through 11. Each C.B.R.

value plotted is the average of two determinations - top and bottom of sample. A wide scatter in results was found over the full range of moisture contents; reproducible C.B.R. values are difficult to obtain at moisture contents close to the optimum for compaction. Lines of equi- C.B.R. are drawn to best fit the individual C.B.R. values on the above mentioned figures. The zero, 5% and 10% air voids are shown, and for ease of reference the standard and modified A.A.S.H.O. compaction curves are repeated.

Once again we refer to soaking C.B.R. samples prior to testing. At the outset, we had intended to soak the samples, but having tried it on the Arigna soil, it was apparent that the soaked sample became so soft that it would not represent a useable subgrade material. The surcharge of 10 lbs. used had little effect in retarding swelling of the Arigna Soil. The R.R.L. Road note 29 states that the most reliable laboratory C.B.R. values are obtained at 5% air voids, and it contends that higher degrees of saturation lead to excess pore pressure beneath the plunger in the C.B.R. mould. It claims that the effect may yield unrealistically low values of C.B.R. for design. The proposal in Road note 29 refers specifically to the soil and weather conditions prevailing in the British Isles, which may not be applicable elsewhere.

3.2 Mobilisation of Internal Friction and Cohesion at Low Strains.

Unlike static triaxial tests for structure foundations where progressive failure is assumed complete at 20% strain, the load indicative of flexible pavement

distress is that which produces 5% strain in the triaxial compression test. (Seed, Chan and Monismith, 1955). Work reported by Schmertmann and Osterberg, (1960) indicated that at low values of principal strain the cohesion is fully mobilised, but appreciable strain is required to develop the internal friction component of shear strength. Now if it can be shown that a soil is non-dilatant, at strains up to 5%, then the operative component of the shear strength is entirely cohesive. Two means of checking that a soil behaves as a purely cohesive material are:

(a) Separate the apparent cohesion and internal friction components with the aid of Mohr diagrams.

(b) Measure the overall volume change of the specimen during shear.

Both means are explored in the static triaxial tests, performed on 1.5" diameter specimens. Typical results are the plots of Mohr circles against strain shown in Figures 14 and 15 and the volume change vs strain in Figure 15.

The plots show that apparent cohesion is actually the component of shear strength mobilised at low strains in clayey soils. The more reliable values of the cohesion are obtained at water contents above the optimum for compaction, which is in fact the final moisture equilibrium encountered in subgrades, i.e. in the regime of optimum to fully saturated. Note that the volume of the sample decreases up to strains of at least 5%, and only after appreciable strain does dilation occur. In class-

ical mechanics such a non-dilatant substance is termed a Prandtl-Reuss material (Druker and Prager, 1952).

The main advantage to be derived from the static shear tests is the simplification of theoretical computations that follow. Once a material is shown to behave purely cohesively, the geometry of slip surfaces takes on the simple configuration of a family of orthogonal circles. When internal friction must be considered the slip surfaces are formed by logarithmic spirals intersected by a set of radial lines of the well known Prantl solution for a c, Ø material in plastic equilibrium; a much more difficult theoretical proposition.

In repeated load tests, the strains observed are generally less than 5% for the magnitude of deviator stress pertinent to road subgrades, hence it is concluded that the component of shear strength mobilised is mainly cohesive. This assumes of course, that the mechanism of strain development is the same in static and dynamic tests. However, significant changes in density during dynamic tests could invalidate the assumption.

The deviator stresses at 5% strain shown in Table I indicate the rapid fall off in strength with small changes in moisture content which appears to be characteristic of boulder clays.

3.3. Modulus of Resilience determined from Repeated Load Tests.

General.

As stated at the outset of this text, the modulus of deformation reported by workers using different test techniques shows a wide variation for even similar soil types. For instance Table 16 of the National Co-operative Highway Research Program Report No. 10, summarises the moduli obtained by a half dozen institutions, for the A.A.S.H.O. Road Test subgrade soil, listing individual determinations between 1,000 and 7,000 p.s.i. The situation regarding modulus of deformation is enunciated in Chapter Three of that report and we quote, "A review of published data on the modulus of deformation of the A.A.S.H.O. Road Test materials reveals that:

(a) Very few such results are available.

(b) There is a wide range of values within any one investigation as well as between investigations. The latter is to be expected, because the modulus of deformation varies with conditions of loading, soil properties, etc., and, moreover, is not unique in it's definition".

In this report the modulus of deformation, otherwise termed modulus of resilient deformation is defined as the ratio of the applied deviator stress to the resulting re-coverable strain occuring under each load in the repeated load triaxial compression test already described. The strain is computed by dividing the axial deformation by the specimen height thus finding an average strain, and ignoring the possibility of non-

uniform strain over the height of the specimen. Evidently Seed uses the same method of computation for resilient modulus.

Previous work indicates that the modulus of resilient deformation varies with method of compaction, density, moisture content, stress level and number of stress repetitions. (Seed, Chan, and Lee, 1963). Their study showed that the degree of saturation and stress level most significantly influence the modulus values of the A.A.S.H.O. subgrade soil and Vicksburg Silty Clay. The effects of thixotropy were also investigated, and from their results it can be postulated that strength regained from this source is virtually eradicated by the first 10,000 stress applications. Further it appears that specimens aged for intervals between 3 and 14 days will yield regular values of the modulus (Figure 9 of the above reference).

Essentially, the results reported herein substantiate the findings of Seed and co-workers. The principal variables in this investigation are the stress level, moisture content and density together with some latitude in ageing of specimens prior to testing. The notable difference in our test routine - that where the axial load was applied in increments on the same sample rather than the single deviator stress per specimen, in Seeds procedure, makes strict comparisons questionable. The single deviator stress would probably yield higher strains for the same magnitude and number of repetitions. Incremental loading, however, is not necessarily at variance with reality since it simulates construction traffic which will condition the subgrade before normal

traffic stresses are imposed. Consequently the resilient moduli obtained in 'incremental tests' may be quite applicable to subgrades in service, in our opinion.

The resilient moduli vs number of stress applications, plotted in the even numbered figures 22 through 56, show some scatter from the mean values represented by the curves. This is largely due to the sensitivity of the modulus to rounding off the values of the strain. In addition the rebound of the soil specimen is often not exactly the same for every cycle of load in the few cycles recorded for each modulus determination shown. Minor fluctuations in applied stresses were accounted for, and further contribute to random scatter of the plotted points.

3.4. Effects of Stress Level on Resilient Modulus.

The resilient modulus is greatly dependent on the stress level. Low stresses yield the higher values of the modulus. Further, the increase in modulus with number of load repetitions is most pronounced at the lower stresses; as the stress level is raised the modulus approaches a stationary value, i.e. decreasing dependence on number of repetitions. Weaker samples gain the greatest strength (per centwise) with number of load repetitions provided the deviator stress is not excessive. The non-linear behaviour of the modulus with deviator stress is apparent from the aforementioned plots. In general it does not seem possible to predict the value of the resilient modulus from a limited number of load repetitions - at least 10,000 load-cycles appear necessary for a reasonable estimate.

3.5. Effect of moulding water content and density on Resilient Modulus.

In undrained triaxial compression tests the soil moisture content has a most significant effect on the modulus measured. The relationship between modulus of resilience and moisture content is shown in Figures 58 through 62. Most samples were tested wet of the optimum moisture of the Standard A.A.S.H.O. compaction test. The rate of decrease in modulus with increasing moisture content is most pronounced in those soils with sharp peak compaction curves as is seen by comparing Dublin soil with the sharp peak compaction curve with the Gortdrum flat compaction curves. Or from another standpoint, the higher the liquid limit and plasticity index, the lesser the rate of decrease in modulus with increasing moisture. Samples compacted dry of optimum and subsequently soaked will show low values of modulus even at high densities; this is inferred from the results of Test No. D5, the one test where soaking of the specimen was undertaken.

3.6. Effect of Confining Pressure on Resilient Modulus.

Confining pressure is not among the variables specifically examined herein. However, in view of several data reporting increase in modulus of deformation: (Hardin and Richart, 1963) according to fractional powers of the confining pressure, it is worth stating that no change in modulus was observed on altering the cell pressure over the range used in the present series of repeated load tests.

The narrow range of confining pressure is perhaps the cause of apparent lack of dependence. In any event, the influence of confining pressure is not so well-defined for clay soils, unlike granular materials, due to pore pressure, consolidation effects, etc. (Ibid 1963).

3.7. Influence of Frequency of Stress Applications and Duration of Pulse on Modulus.

The duration of the stress pulse in subgrades caused by moving wheel loads has been measured at the R.R.L. (Wiffin, 1955). It depends on the speed of the vehicle, the depth of the point of measurement below the road surface, the type of construction and the contact area of the tyre. Actual measurements have shown that the duration of the pulse can vary from 0.2 to 1.0 sec. (according to the R.R.L. investigators). Data, believed to have been obtained at the Stocton Test Track California, indicate that the stresses at the base subgrade interface are virtually constant for speeds over 10 m.p.h., which would imply that the modulus of deformation is not greatly affected by vehicle velocities in excess of 15 m.p.h.

The repeated load tests were performed at 10 stress applications per minute and pulse duration equal to 0.6 seconds. This combination corresponds to a traffic velocity of 10 m.p.h. approximately. The somewhat low frequency and slow load rate was imposed by the need for reliable running of the equipment over long periods. For short term runs the number of stress applications can be raised to 20 per minute and this was done as a matter of checking the effects of frequency on modulus. The result is shown in Figure 63. It indicates that for

the limited number of stress applications at 20 per minute the resilient deformation trace is unchanged.

3.8. Estimation of Modulus of Resilience from C.B.R. Values.

Considering the data presented and the deductions concerning effect of stress level on modulus of resilience, it is evident that attempts to obtain a 'one to one' correspondence between modulus of resilience and C.B.R. is futile. This appears all the more true, when one faces the problem of reproducibility of C.B.R. results. However, overlooking the difficulties mentioned, a plot of modulus of resilient deformation vs C.B.R. is shown in Figure 64. The wide scatter in results is evident. The inset on the figure is a reproduction of data reported by Heukelom and Klomp, 1964, and is included to further emphasise the scatter of individual determinations in any scheme of correlating dynamic moduli and C.B.R. values. The practical importance of repeated load tests could be argued on that basis alone.

3.9. Permanent Deformation of Samples Measured in Repeated Load Tests.

Permanent deformation seated in the subgrade is recognised as the major cause of distress in flexible pavements (Visic and Domaschuk 1964). Excessive plastic flow shows up as ruts that follow the tyre tracks, or as local depressions in the surface. The question arises as to what deformation of the sample in laboratory tests would be indicative of excessive vertical stress at the base course-subgrade interface. In foundation work, the

plastic strain corresponding to incipient failure is 20 per cent of the original specimen height, but for pavements it is considered that 5 per cent strain in static triaxial tests constitutes failure, (Seed and Chan, 1961). In the case of repeated load tests, no commonly accepted value of deformation is specified as indicative of critical stress. On the other hand, unacceptable deflection of the road surface has been set at various quantities in the range 0.1 to 0.2 inch; Burmister (1943) states an upper limit of 0.2-inch for displacement. In practice, reconstruction of the flexible pavements at Alconbury Hill was not necessary until the average deformation had reached 0.5 inch. (Croney and Lee, 1965). It appears from the above statements that stresses producing a permanent deformation equal to 0.2 inch are the maximum that need be investigated in repeated load tests. Consequently, a permanent deformation of 0.2 inch was adopted in the present work.

3.10. Effects of Deviator Stress on Permanent Deformation Measured in Repeated Load Tests.

The plots of permanent deformation vs number of stress applications for different stress levels shown in the odd numbered Figures 21 through 55 indicate that, in general, the permanent deformation is not well ordered function of either the deviator stress or the number of stress applications i.e. the curve shapes are dissimilar for each stress increment. Non-linearity between stress and permanent deformation is evident at virtually every number of repetitions. The permanent deformations were not converted to strains here, because the assumption of uniform permanent strain over the full height of the sample does not appear tenable; soft samples display

enlargement of the ends, while firm specimens bulge at mid-height. The plots bear out the viewpoint that in general, it is not possible to predict the cumulative effect of a series of repetitions of deviator stress of different magnitudes from the permanent deformation recorded for individual stress levels. Nevertheless, certain idealizations of the behaviour are feasible, such as representing the permanent deformation vs number of stress applications for any one deviator stress by a straight line of negative slope on the semilogarithmic plots. For instance the scheme would be reasonable for test number D1, accurate for D4, doubtful for D2, but poor for G3.

3.11 Effects of Confining Pressure on Permanent Deformation.

The effects of confining pressure on permanent deformation were not investigated. One difficulty is that no field measurements are available to confirm that the wave form of confining pressures shown in Figure 65 is in fact a faithful reproduction of the wave form for horizontal stress in the subgrade beneath a moving wheel load. Work on the forthcoming contract will it is hoped elucidate the form of horizontal stress build-up under field conditions. On the other hand, cell pressures in the range 0-10 p.s.i. probably produce only minor effects on axial deformation in repeated load tests.

3.12 Effect of Moulding Moisture on Strength of Compacted Clay in Repeated Loading.

Similar to its influence on the modulus of resilience the moulding water content greatly affects the permanent deformation developed at any value of the deviator stress. Both site and laboratory experience show that most boulder clays are susceptible to ingress of moisture and subsequent softening. The fall-off in resistance to load with increasing moisture content is seen to advantage in Figures 66 through 70. Again the soils with sharp peak compaction characteristics show the most rapid drop in strength as can be seen from the results obtained for Dublin and Gortdrum samples.

Since the first repeated loading triaxial tests were performed by Buchanan and Khuri the existence of threshold stress has been postulated by different workers. Buchanan and Khuri (1954) Kawakami and Ogawa (1963) Larew and Leonards (1962). Further, the existence of a critical vertical stress level is suggested by field measurements of surface rut depths vs vertical stress on subgrade reported by Vesic and Domaschuk (1964). Surprisingly, no concerted effort to apply the threshold stress concept has been reported.

The threshold stress is that stress below which no significant permanent deformation occurs under repeated loading. Evidently the curves of equi-deviator stress projected back to the horizontal line through the origin in Figure 66 through 70 yield the threshold stresses vs water content. Thus, as an example, the threshold stresses for the Arigna boulder clay are estimated at 4, 8 and 12 p.s.i. at 13, 12, and 10.7 per cent moisture respect-

ively for 50,000 stress applications (see Figure 66). The threshold stresses so derived should serve as a good indication of the maximum permissible vertical stress at the base-subgrade interface, in the writer's opinion. The fact that vertical stresses in the subgrade are known to fall off rapidly with increasing depth below the interface (Ahlvin and Ulery, 1962) supports the hypotheses. The vertical stress that does not produce appreciable permanent deformation in an 8-inch column of soil in the repeated load test is unlikely to develop serious displacements in a subgrade - a continuum that may be likened to stacks of short columns of which the uppermost few are stressed. In this sense, serious displacement is meant to convey structural failure of a flexible pavement, a situation defined in the National Co-operative Highway Research Programme, Report No. 10 as, "a state in which repeated application of a specified wheel load results in ever increasing plastic deformations of the pavement surface." However, recourse must be made to experience gained by field measurements of subgrade stress in typical pavement structures to apply the threshold stress concept due to conflicting findings found when comparing theoretical and measured deflexions.

In practice, however, it may prove uneconomical to limit subgrade stresses to values as low as the threshold between predominant elastic and elastoplastic behaviour. The requirement then is for a theory that predicts adequately the permanent set in, and the displacement of, the elements of material of pavement layers and subgrade soils in the stressed zone. No easily applied theory to meet this requirement has emerged so far. The supplement to this report (Part II) will delve further into this matter of theory in

the elastoplastic range.

To summarise, the threshold stress is derivable indirectly from a set of results of repeated load tests performed on samples covering a practical range of moisture contents. The density associated with the moisture content and threshold stress will lie in the domain bounded by the Standard and Modified A.A.S.H.O. compaction curves for ordinary field compaction. The correct choice of threshold stress from plots demands an estimate of the equilibrium moisture content of the subgrade, and probable traffic density. Equilibrium moisture content may be difficult to assess, but this should not detract from the usefulness of the threshold stress concept. The threshold stresses were derived herein at 50,000 repetitions, but such an arbitrary choice would not be acceptable for every traffic intensity. Further, the numerical value of the resilient modulus obtained at the threshold stress is deemed to be a significient parameter in the application of the elastic theories (Boussinesg's and Burmister's multiple layer) to analysis of pavement structures. Measurement of strain in-situ as undertaken in A.A.S.H.O. and Alconbury Hill road tests provides essential background information for practical application of the ideas discussed in the foregoing paragraphs.

3.13. Effects of Frequency and Waveform on Permanent Deformation.

The frequency and duration of the stress pulse have complicated effects on the deformation of clay in repeated load tests (Seed and Chan 1961). Different combinations of such factors as creep with time under load, re-separation of particles during periods of unloading, thixotropic strength regain in extended rest periods, and rate of application of load are put

forward as causes for anomalous behaviour of clays. Undoubtedly, creep under sustained load and rate of loading are major factors. Numerous papers have been published on the various aspects of creep in soil. Suffice it to remark that a stress waveform with a flattened peak will increase deformation. The work of Whitman (1957) shows that cohesive soils exhibit increased resistance to deformation as the rate of loading is accelerated.

The waveform of axial stress shown in Figures 19 and 63 is of similar shape to the waveforms observed under moving traffic by Wiffin (i.e. symmetrical with a sharp peak). The slight ripple occuring between the pulses originates in the oscillations of the spring yoke, and the small negative peak is due probably to delay in the action of the permanent deformation compensator; neither would appear significant in contributing to permanent deformation. As stated previously, the pulse duration corresponds to low vehicle velocity (10 m.p.h.) therefore the deformations measured are hardly less than what would develop in the same size element of subgrade under similar stress conditions. The short run at 20 cycles per minute was not extended sufficiently to produce a noticeable change in the pattern of permanent deformation. The deformation trace in Figure 19 shows clearly a recovery period of the order of 2 secs. for the rebound of the sample.

It is worth noting that in many of the plots of axial deformation vs number of stress applications reported by Seed, the strains appear to take on greater values than are reported herein (Seed et alia, 1955,

1962). The disparity may be attributed to different stress waveforms i.e. sharp peak vs sustained peak load. (Seed's waveforms tend to be steep-fronted with a constant peak value maintained for some interval before unloading (Seed, Chan and Monismith, 1955, Seed and Fead 1959).

Summarising, the pulse duration, frequency and waveforms are significant factors in studies of permanent deformation under load. In this investigation the frequency was rather low and pulse duration was correspondingly longer than those associated with normal traffic flow. Axial stress waveform was close to the ideal. Nevertheless, the discrepancies are not so serious as to invalidate the application of the results to real subgrade situations, in the writer's opinion.

3.14. Relationship between Sample Resistance in Repeated Loading and Unconfined Compression Test.

A search for relationships between results of repeated loading and static tests is a matter of considerable importance for the following reasons:

(a) Empirical methods of pavement design make extensive use of the results of static strength tests e.g. C.B.R. and triaxial shear.

(b) Effort and expense would be lessened if the results of relatively few repeated load tests could be extended by established relationships between them and the conventional tests.

One approach to the problem is to compare the deviator stresses required to produce arbitrarily selected, or limiting, deformations in dynamic and static tests. Assume, for the purpose of this discussion, the unconfined compression tests indicate the stress at 5 per cent strain that will produce structural failure in pavements, then, from considerations stated at the beginning of this section, the deviator stress that causes 0.1" permanent deformation in the repeated load test may be an acceptable equivalent. The comparison is drawn in the following tabulation of deviator stresses measured in unconfined compression tests performed on specimens that had been tested earlier in repeated loading.

TABLE 2

Comparison of Deviator Stresses in Repeated Load Tests and Unconfined Compression Tests.

Repeated load test sample	Repeated load test		Unconfined Compression test.	Ratio of Deviator Stresses
	Deviator stress p.s.i.	Permanent Deformation inch	Deviator stress p.s.i. (@ 5% strain)	
E 1	4.0	0.1	23.0	5.7
E 2	8.0	"	48.6	6.0
S 1		"	51.7	
S 2	3.3	"	18.8	5.7
S 3	8.5	"	30.2	3.8
G 1	17.5	"	44.9	2.5
G 2	8.0	"	44.2	5.5
G 3	3.0	"	16.4	5.5

These comparisons show the mean ratio of unconfined compressive stress to repeated deviator stress is 5:1 approx. Consequently, if the unconfined compressive stress taken at 5% strain is divided by 5 the deviator stress producing 0.1" in an 8" specimen in repeated loading is found. The conclusion is based on very few data, and consequently is not a proven relationship for clays in general. However, the approach merits further attention as more data becomes accumulated.

3.15. Influence of Atterburg Limits on Behaviour of Soils in Repeated Loading.

No quantitative assessment of the influence of Atterberg limits on results of repeated load tests were established. This is due, in part at least, to insufficient data for reliable correlation. However, from a qualitative standpoint, it may be stated that soils with liquid limits plotting between 35 and 45 and above the A-line on the Casagrande Plasticity Chart behave the most satisfactorily under repeated stresses. The statement is difficult to qualify in the case of highly plastic clays (L.L. over 50) as repeated load tests indicate high resistance to deformation, but experience in the field shows that 'heavy' clays are problem subgrades in many instances. On the other hand clays of low plasticity behave more or less like silts and consequently exhibit higher rebound particularly at high degrees of saturation. Further work is necessary to correlate Atterberg limits with soil deformation in dynamic tests.

3.16. Influence of Ageing of Samples on Behaviour of Soil in Repeated Loading

The influence of ageing is most marked at low values of deviator stress, but it is virtually eliminated by high deviator stresses and large number of stress repetitions. Thixotropic strength regain could perhaps account for the inconsistent results obtained in the repeated load tests on sample numbers A 1 and S 3.

Chapter 4

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

4.1. Summary

To start with, we developed equipment suitable for testing comparatively large specimens under repeated loading in triaxial compression. This equipment has been fully described.

Five glacial till soils (minus 3/4") were subjected to repeated load tests. Time allowed only a limited number of tests on each soil.

The primary quantities, elastic and permanent axial deformation, were isolated in the measuring system. The problem of measuring the lateral deformation of the sample, using a displacement transducer inserted in the triaxial specimen, was investigated; it appears that with some further development this will be satisfactory.

We tried to measure the pore pressure during testing, but reliable pore pressure measurements on partially saturated soils are difficult. Further work will be needed to produce a satisfactory technique; lags in response and deairing are troublesome in such measurements.

The results of repeated load tests were analysed with respect to the requirements of flexible road

pavement performance. In addition, the compaction characteristics, C.B.R. values, shear strength parameters and Atterberg limits were determined.

4.2 Conclusions

We conclude from the investigation:-

1. The modulus of resilient deformation is most influenced by the deviator stress level, moisture content and ageing of specimens before test.

   The maximum values of the modulus were obtained at the lower deviator stresses; a rapid increase in modulus occurring with increasing number of stress applications.

   With increase in deviator stress the modulus became less dependent on the number of repetitions, and approaches are almost stationary value for a particular value of deviator stress.

2. Correlation of dynamic moduli with C.B.R. values is questionable due to the above mentioned dependence of moduli on applied stress. However the method of other authors has been adopted to bracket the modulus vs C.B.R. values.

3. The records of permanent deformation vs deviator stress yielded the most interesting information.

   A method for deriving the threshold stress between elastic and inelastic behaviour of soil vs moisture content is presented.

The analysis relating threshold stress to water content, when extended further, should prove useful in determining the allowable upper limit of the vertical stresses that can be safely imposed on a subgrade.

4. The results show that it is not possible to derive the cumulative effects of different values of deviator stress from a repeated load test using only a single value of deviator stress.

   The pattern of permanent deformation may be quite different in various parts of the plots of deformation vs repetitions of stress. This means that it is not possible to extrapolate deformation from that which is obtained after a few thousand repetitions of stress. In general, the rate of deformation decreases with increasing number of stress repetitions, in a manner similar to the mechanical stabilisation of a gravel road by the action of a large number of wheel loads.

5. The permanent deformation is closely dependent on the moisture content of the sample. The effect of moisture content is most pronounced in those soils with sharp-peak compaction curves.

6. Some work is presented on correlating permanent deformation in the repeated load tests with the results of unconfined compression tests. In this respect the selection of comparative criteria is difficult.

We consider that we have, in the limited time available and as a result of the financial support provided by the contract made a promising start towards furthering the understanding of the behaviour of soil subjected to dynamic loading.

Some interesting results have been obtained; and some useful tentative conclusions have been drawn.

It is our opinion, that there is a strong case of continuing and extending this work in the laboratory in a future contract as well as extending it in the field as is the intention in the renewal Contract just starting.

The aim of an increased programme of testing would be to obtain results from a wider range of soils. The result of this would be to verify our tentative conclusions and, it is hoped, enable us to formulate a method of pavement design which is based on real measureable soil properties.

Appendix I

MINERALOGICAL ANALYSES OF SOILS

The mineralogical composition of the soils were determined at the Laboratory of the Agricultural Institute of Ireland. The results reported by the Institute are tabulated below.

The soil samples were size fractionated to facilitate mineral identification. The grain size distribution was:

Percentages of size fraction present

Sample	Gravel >2 mm	Sand 2 mm - 50 µ	Silt 50 - 2 µ	Coarse Clay 2 - 0.2 µ	Fine Clay <0.2 µ
ARIGNA	46.0	16.9	17.8	12.2	5.5
GORTDRUM	28.6	22.5	28.9	13.9	5.1
DUBLIN	45.4	15.8	25.3	10.3	3.2
SLANE	3.6	33.6	30.6	23.4	6.6
ERNE	20.7	13.7	32.8	22.5	6.4

The Mineral Analysis of the different fractions was:

Sample and Fraction	Calcite:	Quartz	Feldspar:	Mica	Kaolinite:	Vermiculite	Montmorillonite	Chlorite:
ARIGNA								
Gravel > 2mm	21	70		4				
Sand 2 mm - 50μ	39	50		6				
Silt 50 - 2μ	37	45	4	10	3			
C. Clay 2- 0.2μ	5	15	2	27	50	5	5	
F. Clay <0.2μ				34	10	30	20	5
GORTDRUM								
Gravel > 2 mm	13	80	4	6				
Sand 2 mm - 50μ	28	60	4	9				
Silt 50u - 2μ	36	50	4	14				
C. Clay 2 - 0.2μ	1	2		59	5	20	10	
F. Clay < 0.2μ				58	5	20	20	
DUBLIN								
Gravel > 2 mm	36	55	4	5				
Sand 2 mm - 50μ	37	50	4	8				
Silt 50 - 20μ	20	50	8	16				
C. Clay 2 - 0.2μ	1	5	2	56	5	20	10	
F. Clay <0.2μ				60	5	15	15	
SLANE								
Gravel > 2 mm		85	4	10				
Sand 2 mm - 50μ		80	4	14				
Silt 50 - 20μ		50	4	22	2	10	5	
C. Clay 2 - 0.2μ				39	5	40	15	
F. Clay <0.2u				24	5	20	30	10
ERNE								
Gravel > 2 mm	44	40		14				
Sand 2 mm - 50μ	70	12		17				
Silt 2 - 0.2μ	64	10		26				
C. Clay 2 - 0.2μ	41	2		29	10	5	5	10
F. Clay <0.2μ	1			57	5	20	10	

The dark grey colour of the Arigna soil is due to carboniferous limestone (limestone containing colloidal particles of black carbon). The yellow colour of the Slane soil is from ferric oxides (Fe_2O_3) coating on the soil particles. The Gortdrum soil has similar iron oxide staining but to a lesser degree. The Erne and Dublin soils have a grey limestone colouring with some iron oxide staining.

The estimates are based on X-ray diffraction and chemical analysis.

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GLOSSARY

Modulus of Deformation:	The ratio of the applied deviator stress to the resulting strain occuring under each load in repeated load triaxial compression test, or other dynamic strength tests.
Resilient Modulus:	The ratio of the applied deviator stress to the resulting recoverable strain.
Stiffness Modulus:	Same as Modulus of Deformation.
Sensitivity:	The ratio of the undisturbed strength to its remoulded strength, both strengths being determined in undrained tests at natural moisture content of the clay.
Thixotropy:	Process of softening caused by remoulding, followed by a time dependent return to the original harder state.
Threshold stress:	That magnitude of stress where a deformable solid first shows inelastic behaviour.
Elastoplastic:	In this report, elastoplastic denotes behaviour of materials with both recoverable and permanent strains under stresses less than the yield stress of the material.
Pavement Structure:	Includes all the pavement layers placed on the natural subgrade.
Electro-osmoses:	Transfer of moisture through soil under action of electric potential.

FIG. 1.

IAP OF IRELAND SHOWING LOCATIONS OF SOIL SAMPLES

FIG. 2

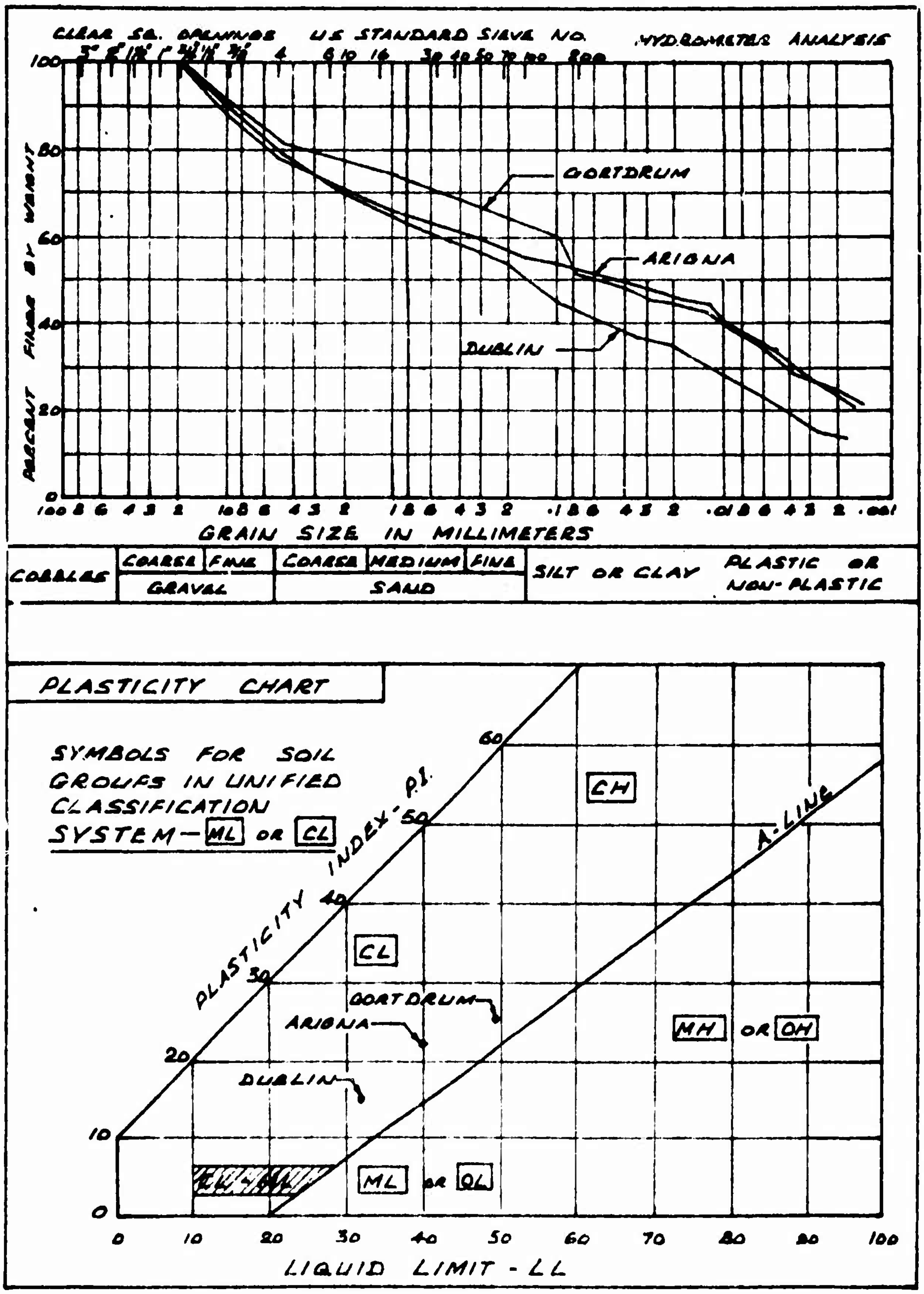
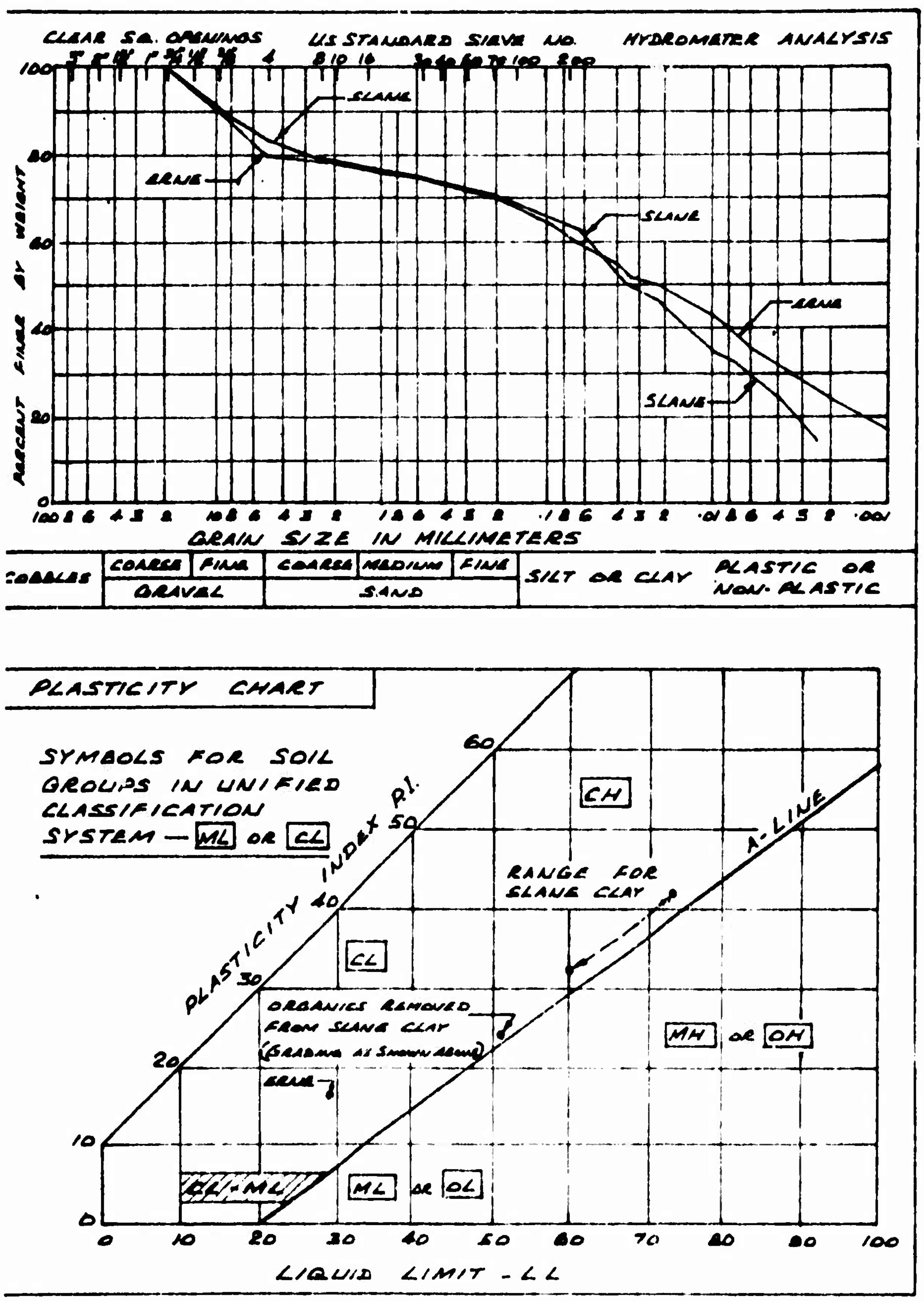
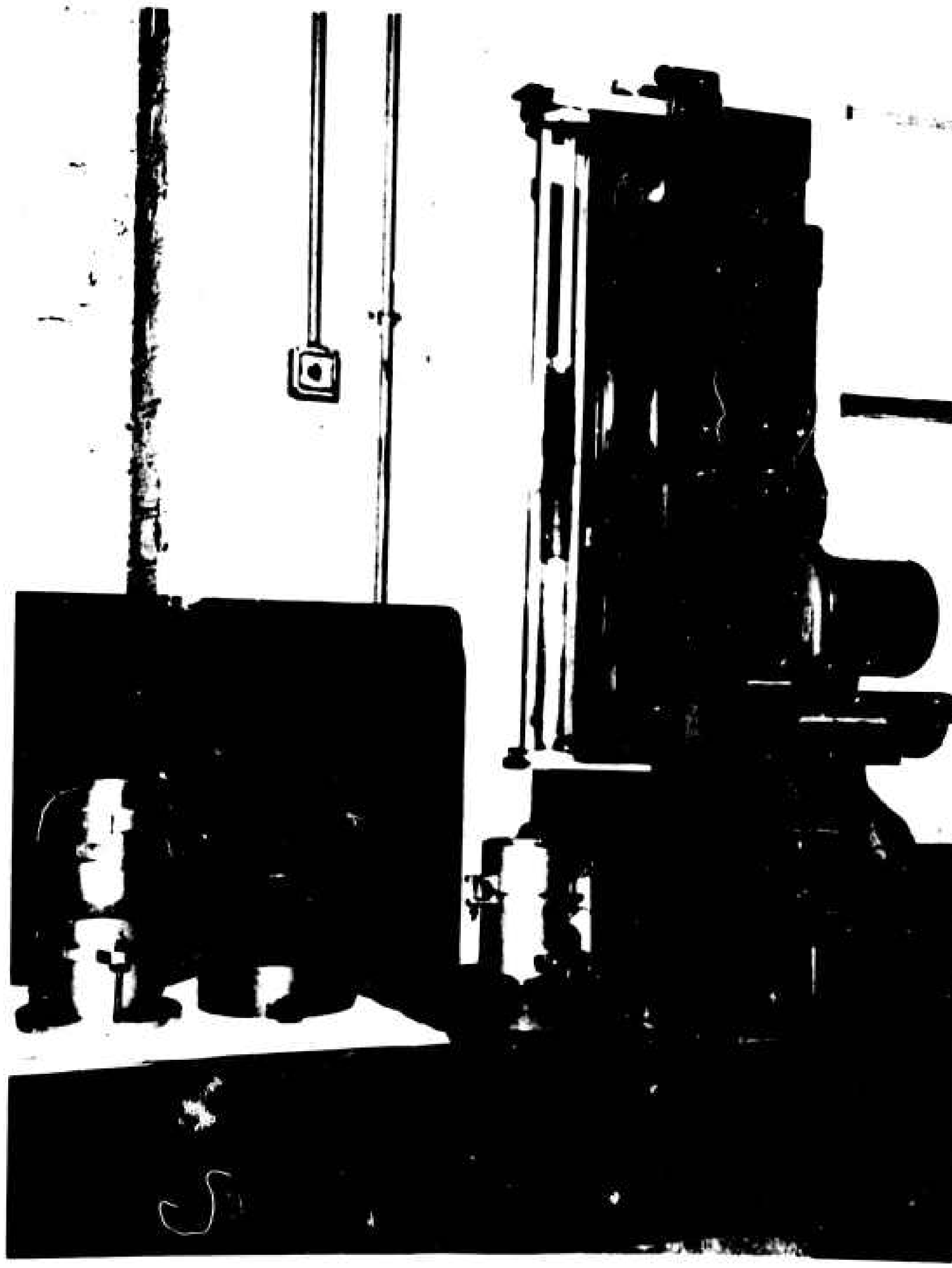

CLEAR SQ. OPENINGS
US STANDARD SIEVE NO.
HYDROMETER ANALYSIS
PERCENT FINER BY WEIGHT
GOORTDRUM
ARIGNA
DUBLIN
GRAIN SIZE IN MILLIMETERS
COBBLES
COARSE
FINE
GRAVEL
COARSE
MEDIUM
FINE
SAND
SILT OR CLAY
PLASTIC OR NON-PLASTIC
PLASTICITY CHART
SYMBOLS FOR SOIL GROUPS IN UNIFIED CLASSIFICATION SYSTEM — ML OR CL
PLASTICITY INDEX - P.I.
CH
A-LINE
CL
GOORTDRUM
ARIGNA
DUBLIN
MH OR OH
ML OR OL
LIQUID LIMIT - LL

FIG. 3



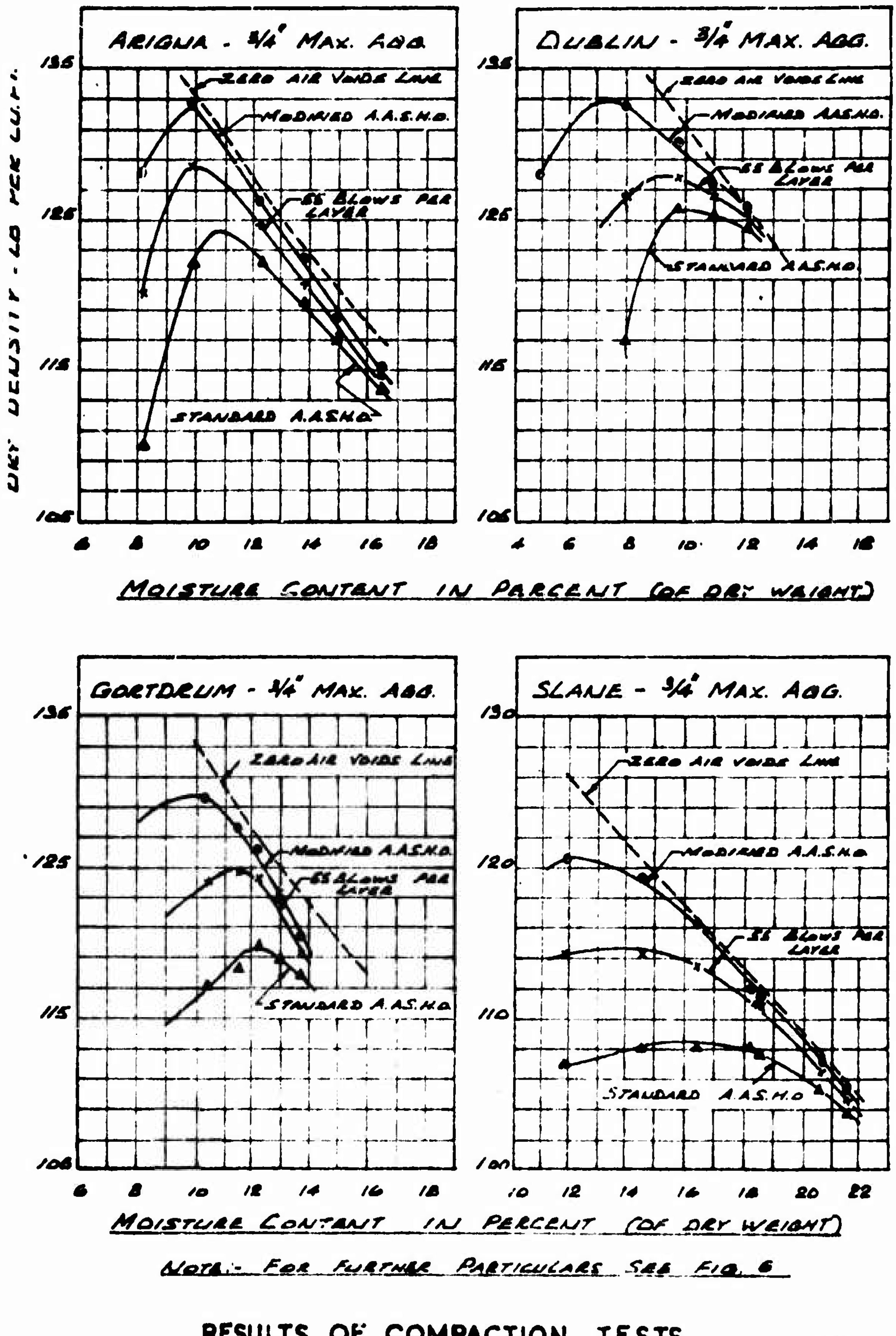
RESULTS OF SOIL CLASSIFICATION TESTS

FIG 4



COMPACTION MACHINE AND MOULDS

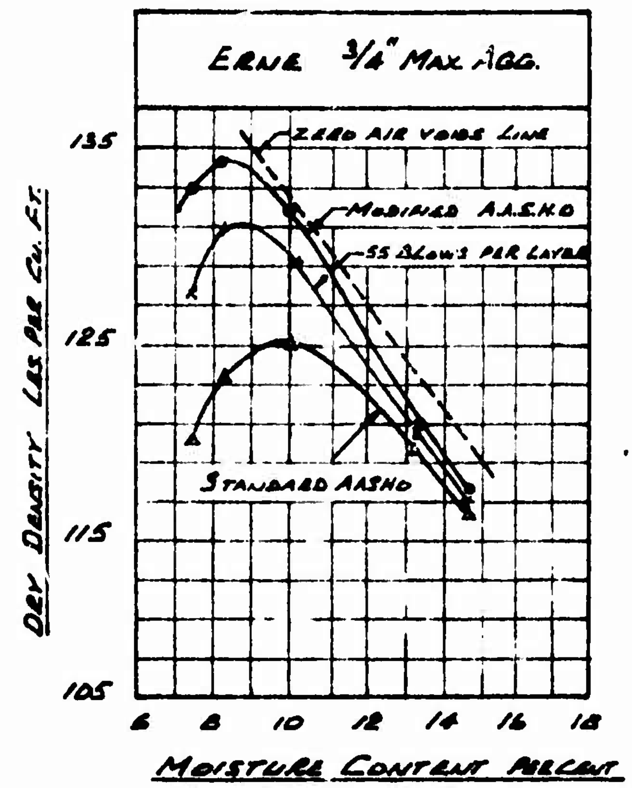
FIG. 5

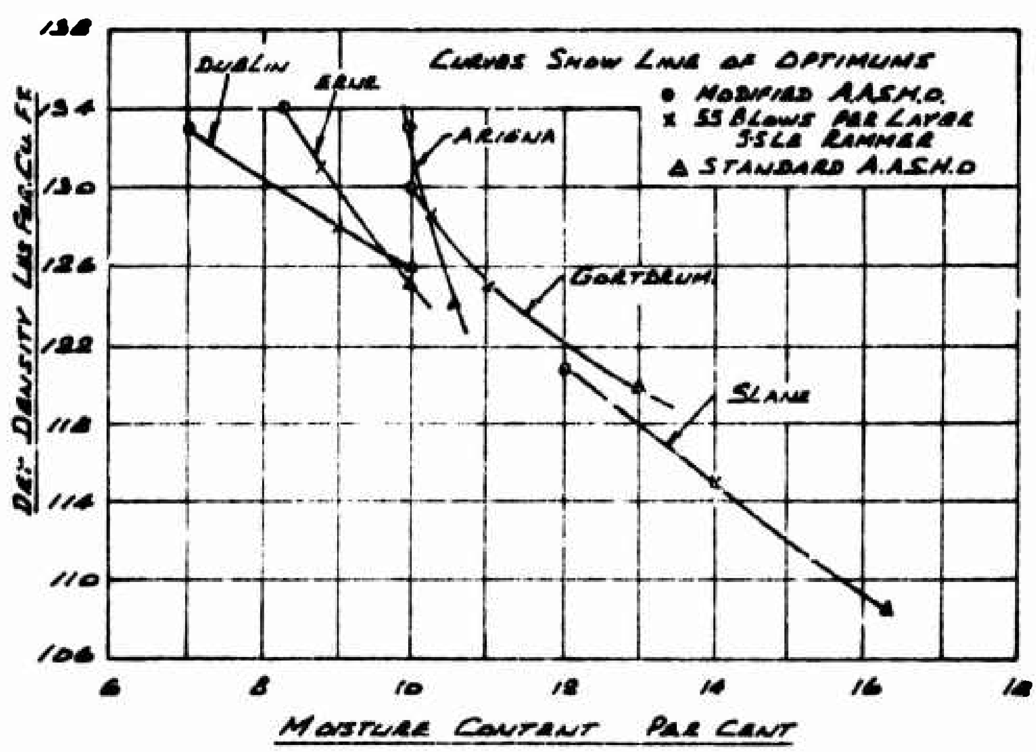


NOTE:- FOR FURTHER PARTICULARS SEE FIG. 6

RESULTS OF COMPACTION TESTS

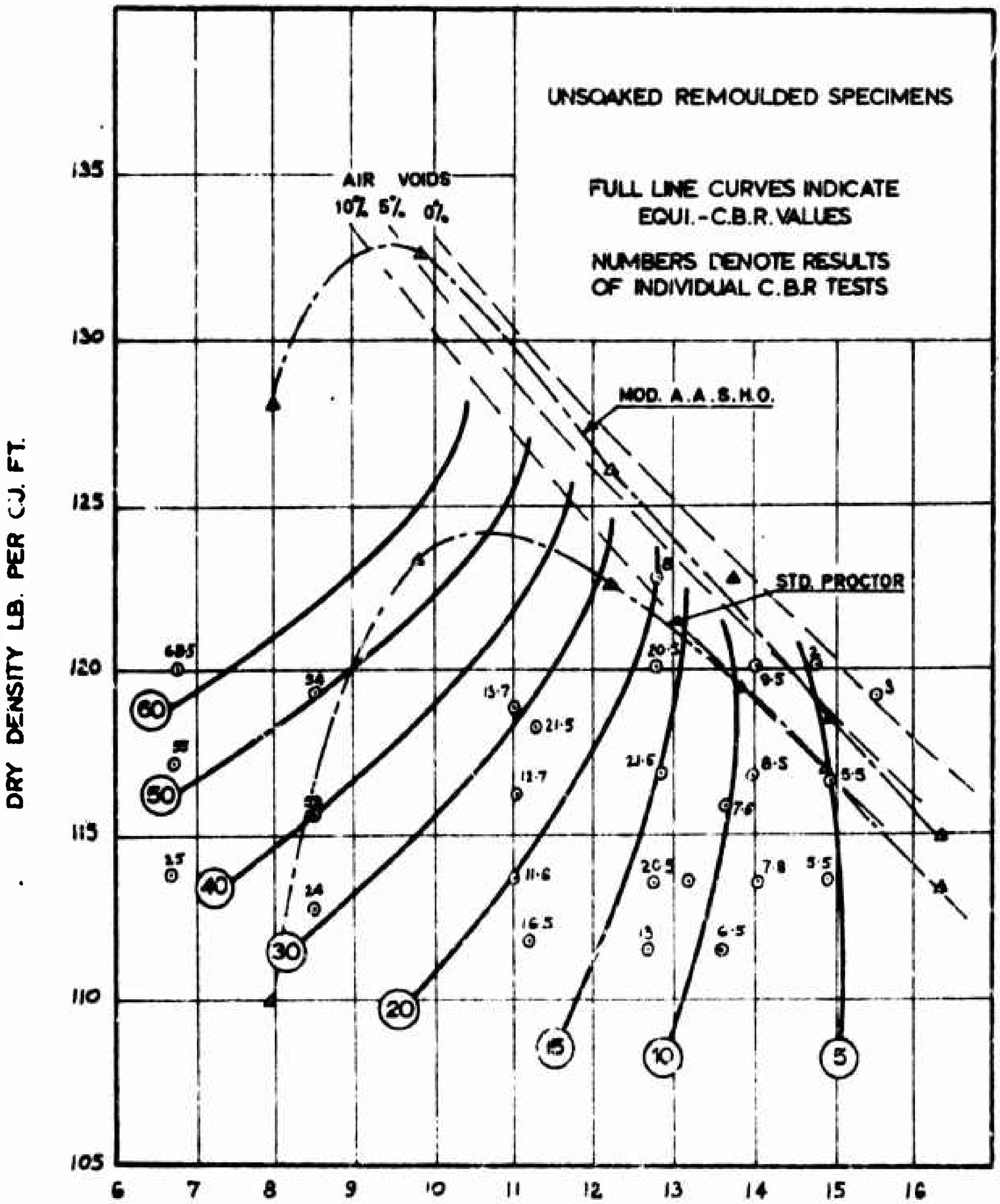
FIG. 6





RESULTS OF COMPACTION TESTS

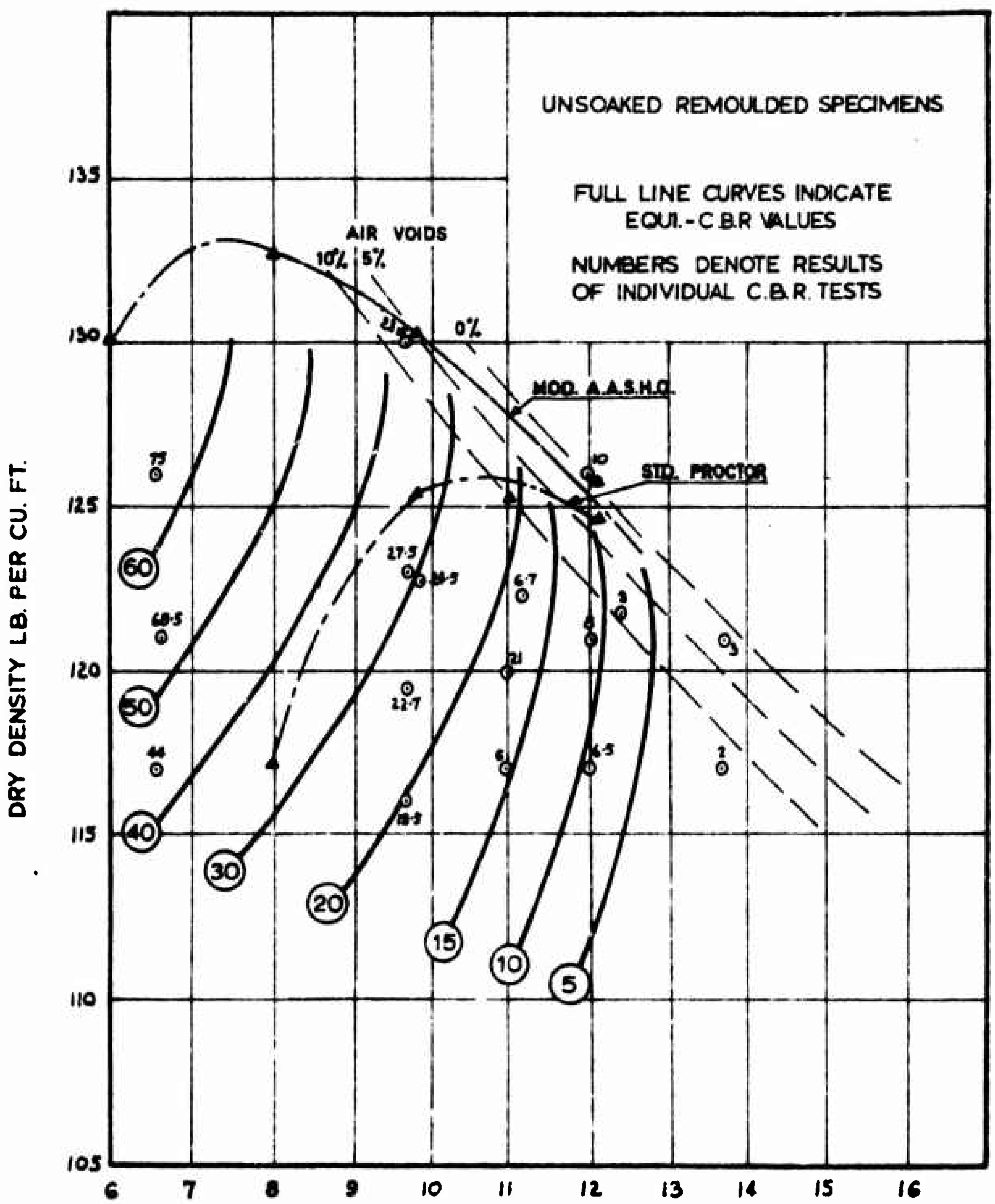
FIG. 7



RELATIONSHIP BETWEEN C.B.R. MOISTURE CONTENT, AND DRY DENSITY

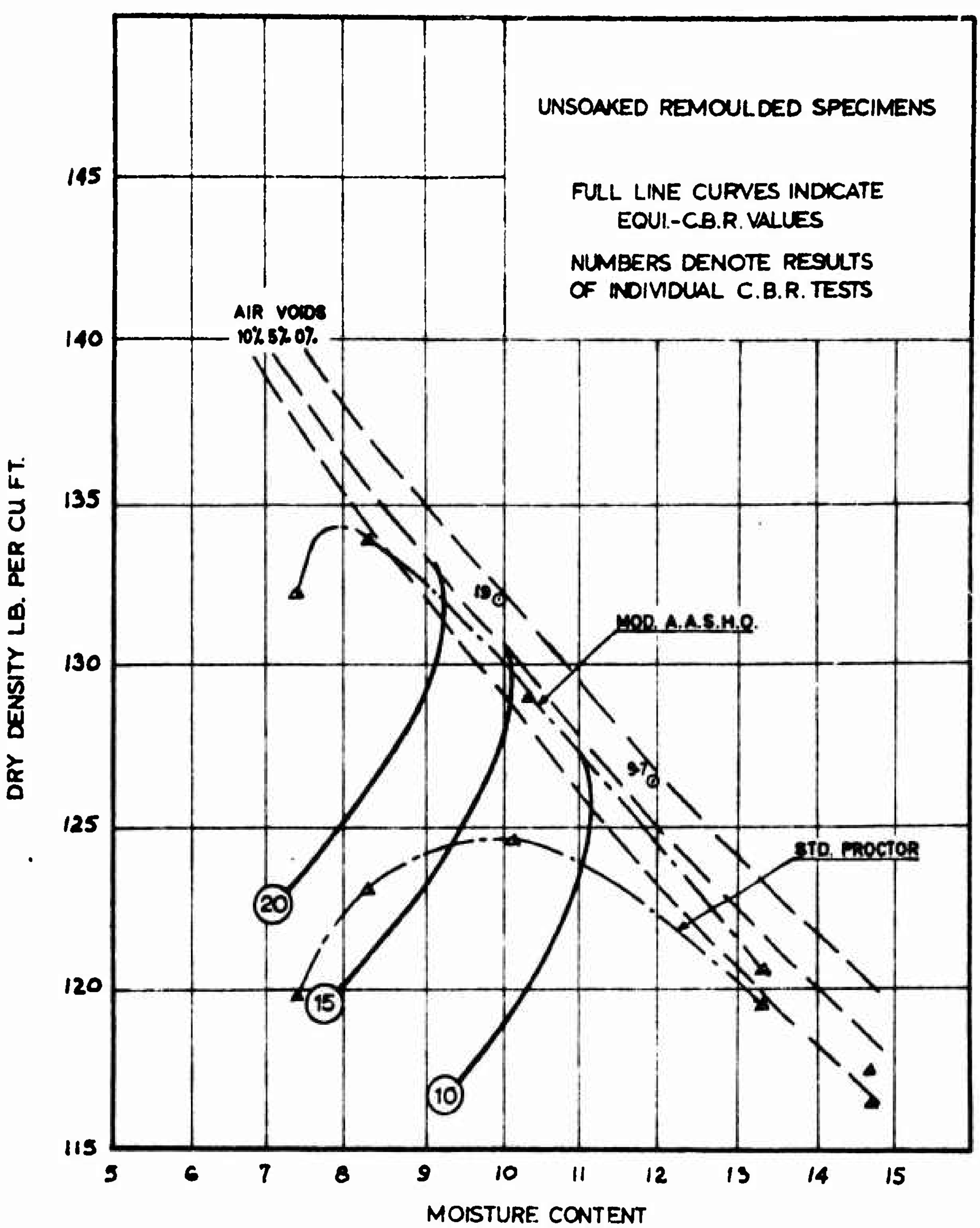
ARIGNA BOULDER CLAY

FIG. 8



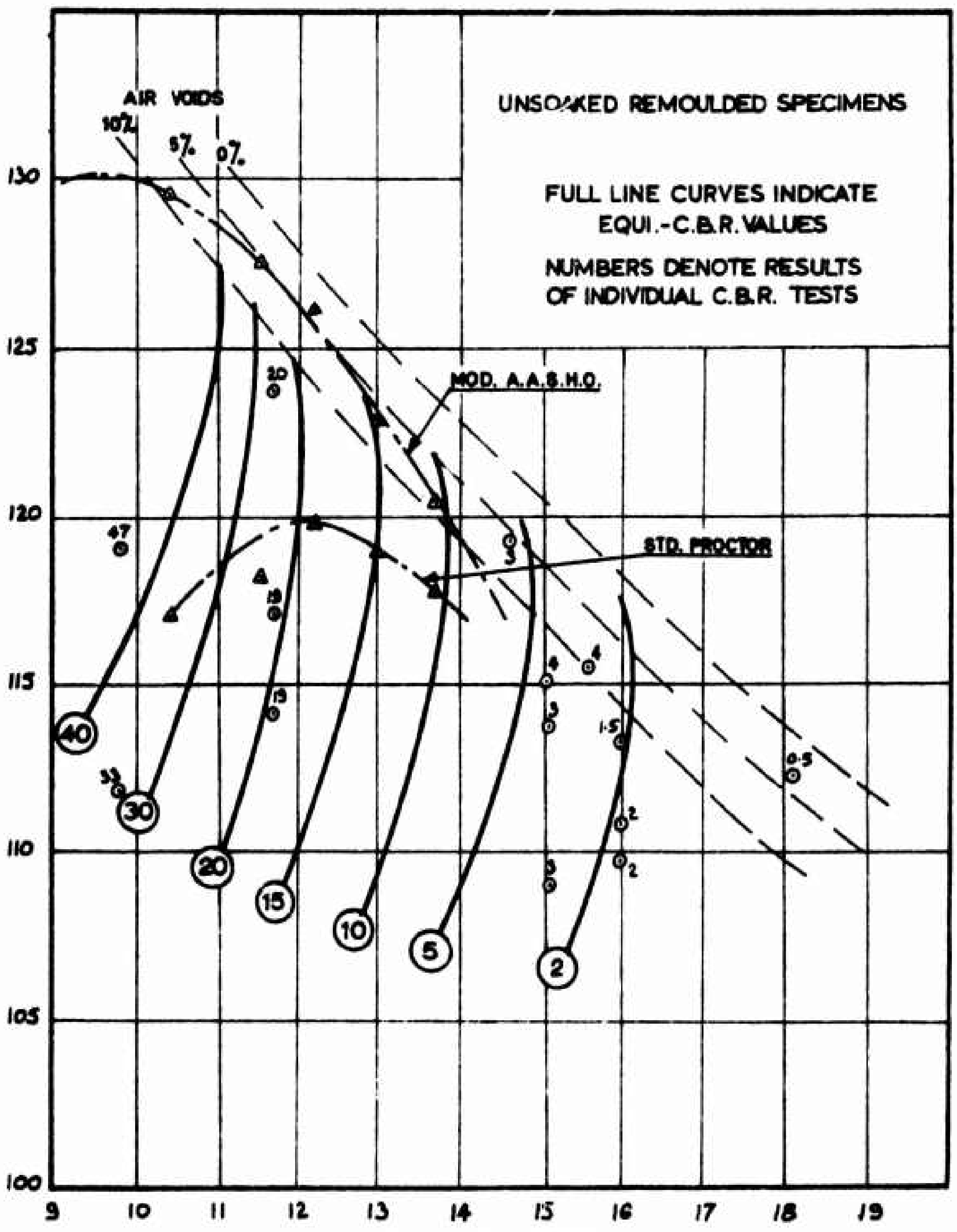
RELATIONSHIP BETWEEN C.B.R., MOISTURE CONTENT, AND DRY DENSITY
DUBLIN BOULDER CLAY

FIG. 9



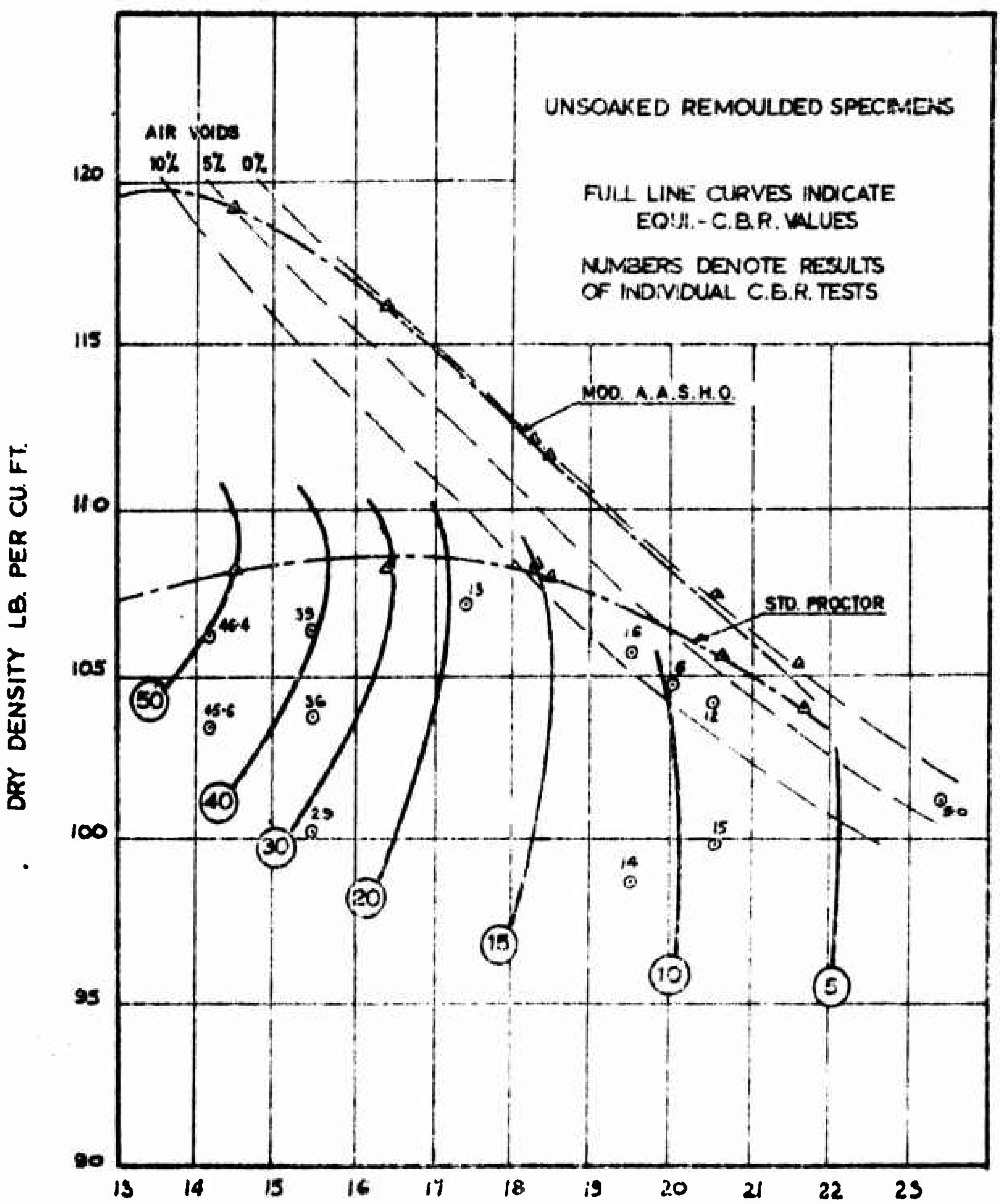
RELATIONSHIP BETWEEN C.B.R., MOISTURE CONTENT, AND DRY DENSITY
ERNE BOULDER CLAY

FIG. 10

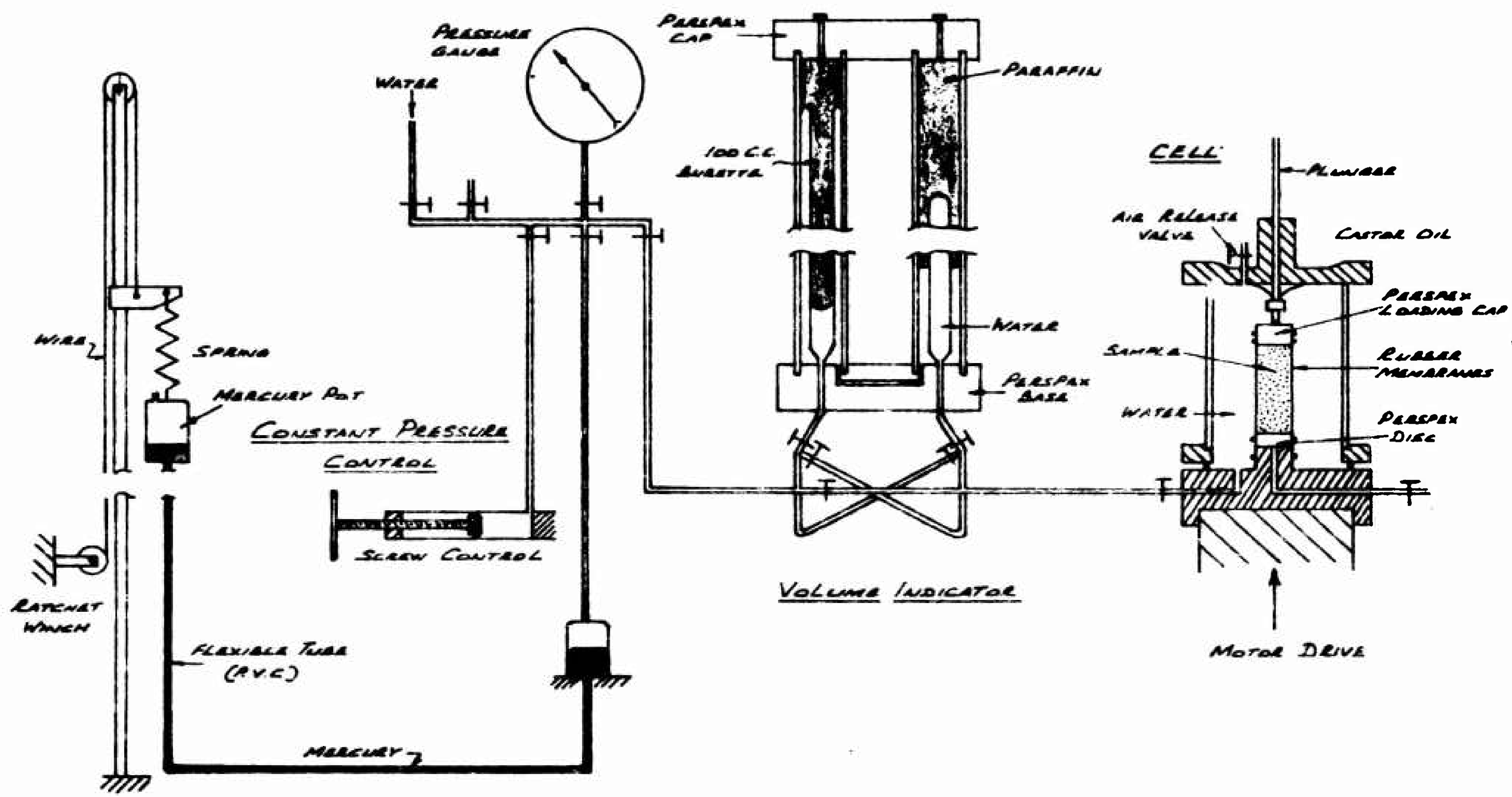


RELATIONSHIP BETWEEN C.B.R., MOISTURE CONTENT, AND DRY DENSITY
GORTDRUM BOULDER CLAY

FIG. 11

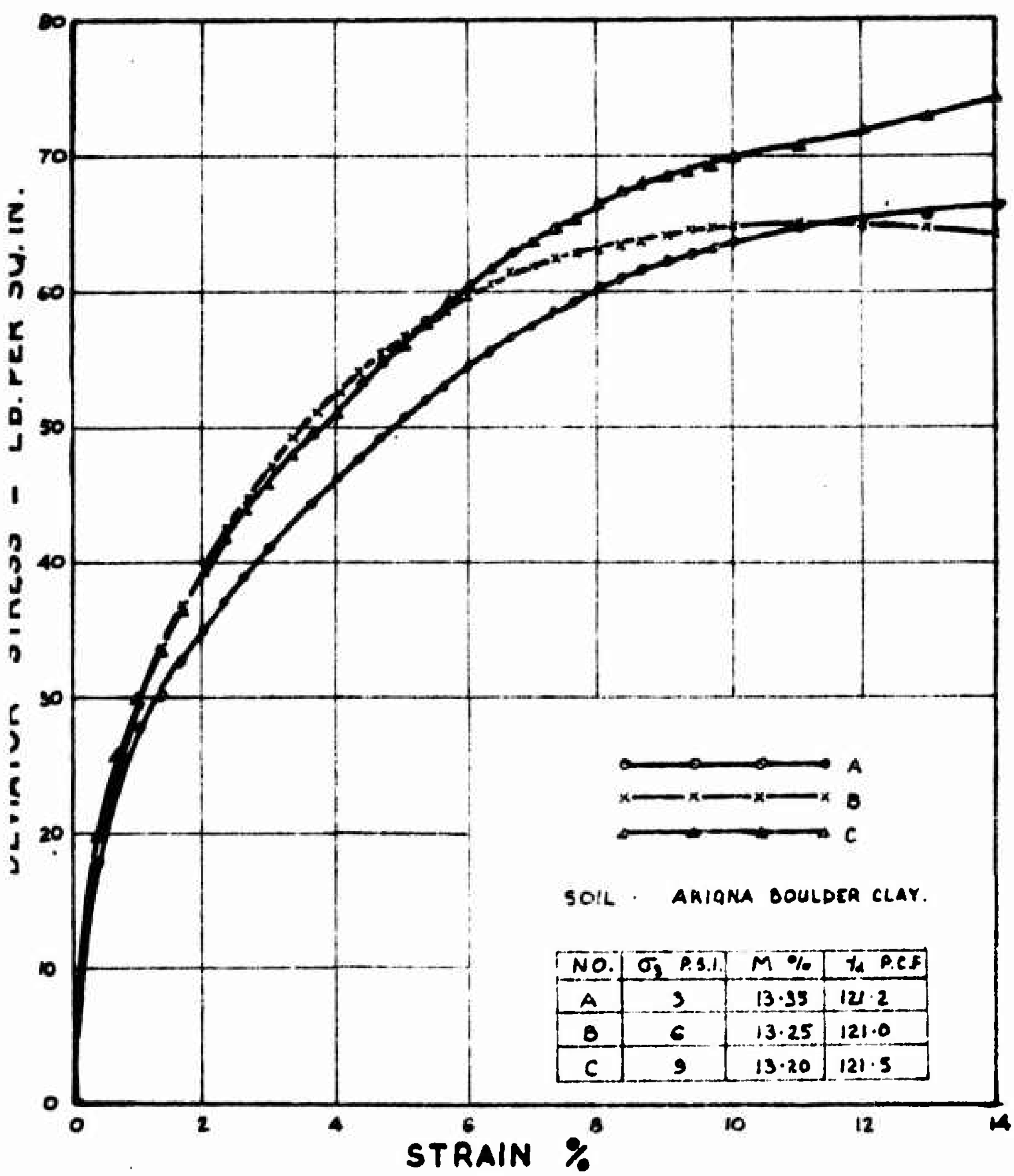


RELATIONSHIP BETWEEN C.B.R., MOISTURE CONTENT, AND DRY DENSITY
SLANE BOULDER CLAY



LAYOUT OF APPARATUS FOR SLOW TRIAXIAL TESTING FIG. 12

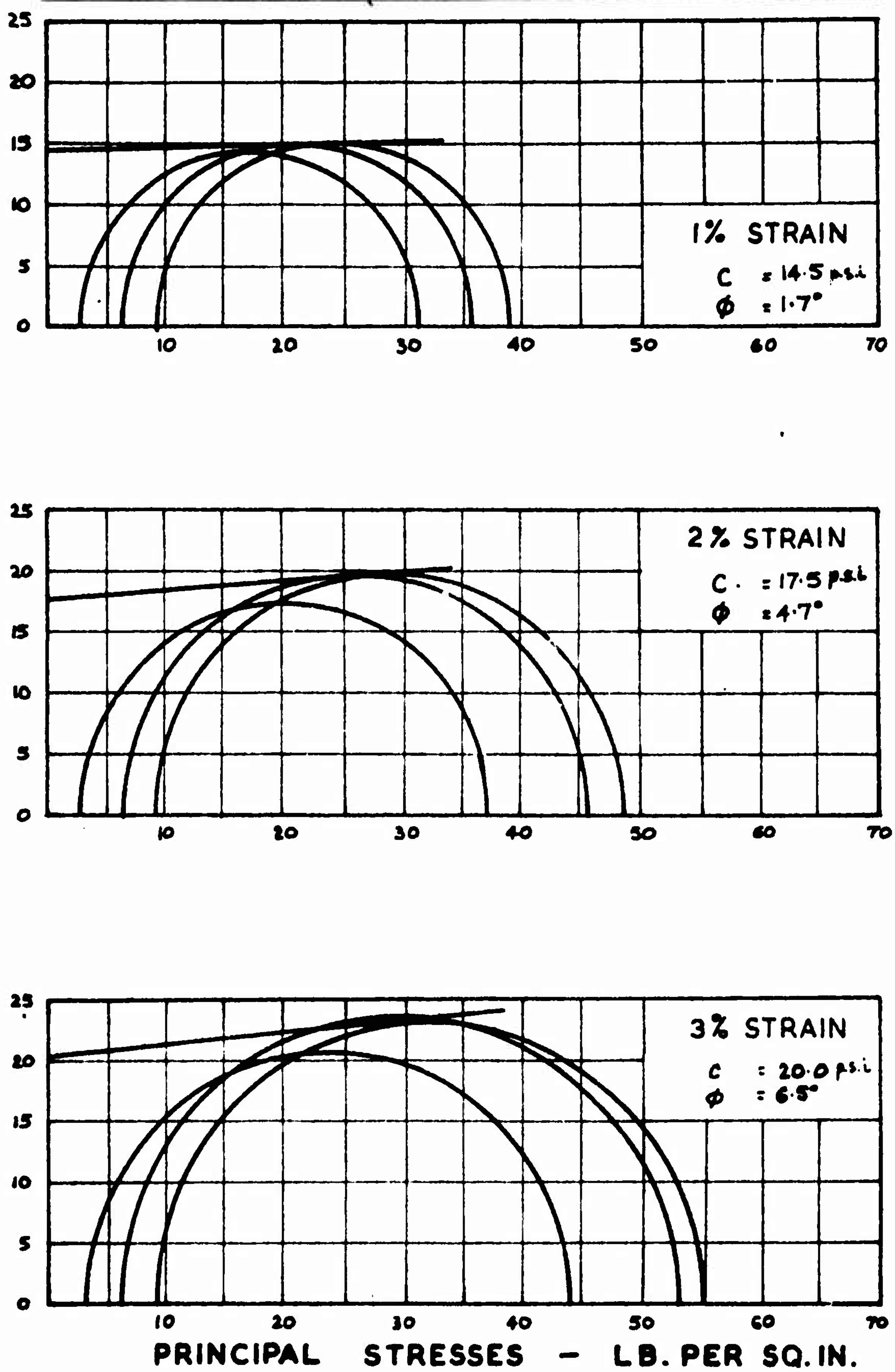
FIG. 13



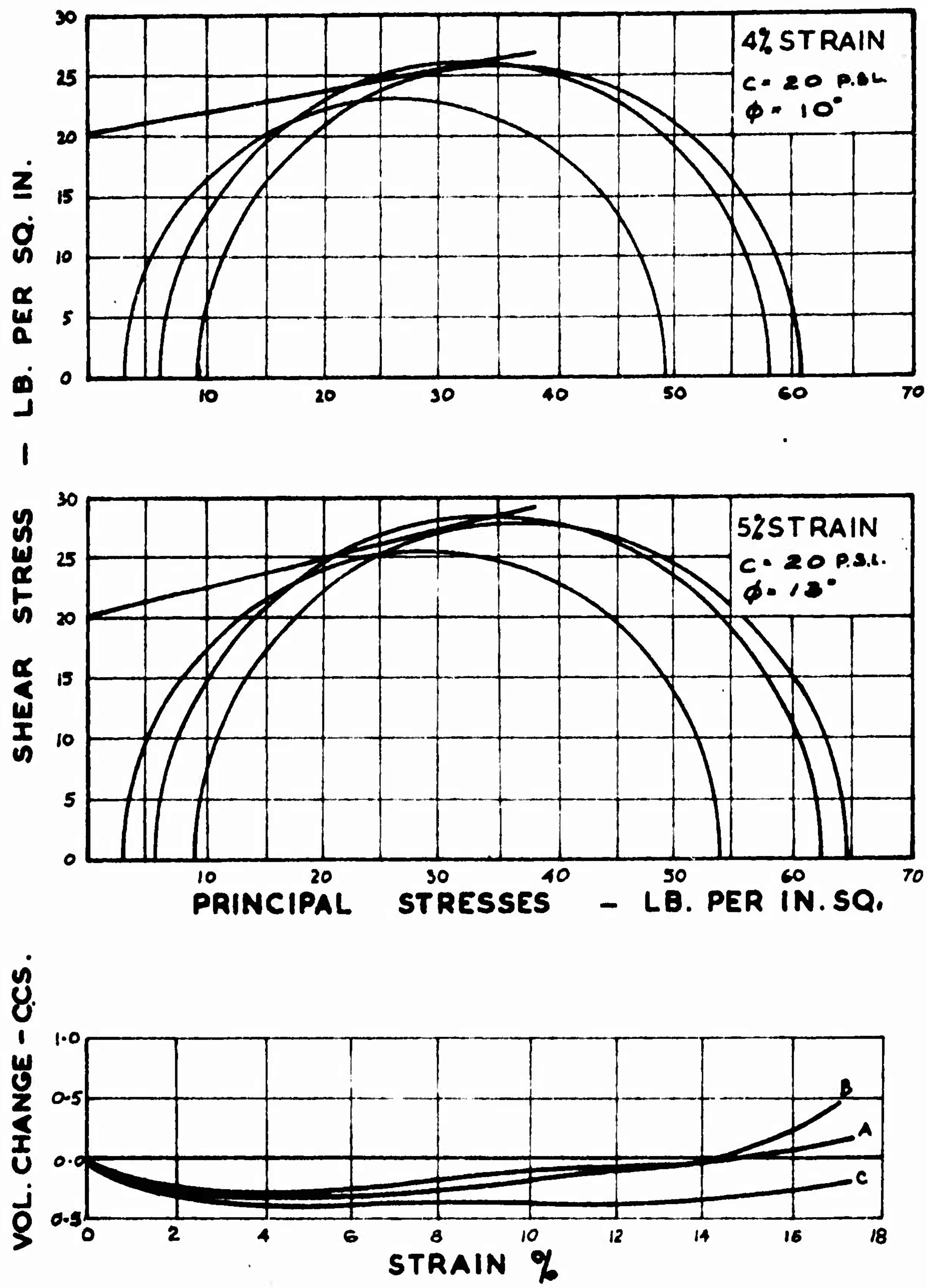
TYPICAL RESULTS OF SLOW UNDRAINED TRIAXIAL TEST

STRESS vs. STRAIN

TYPICAL RESULTS OF SLOW UNDRAINED TRIAXIAL TEST



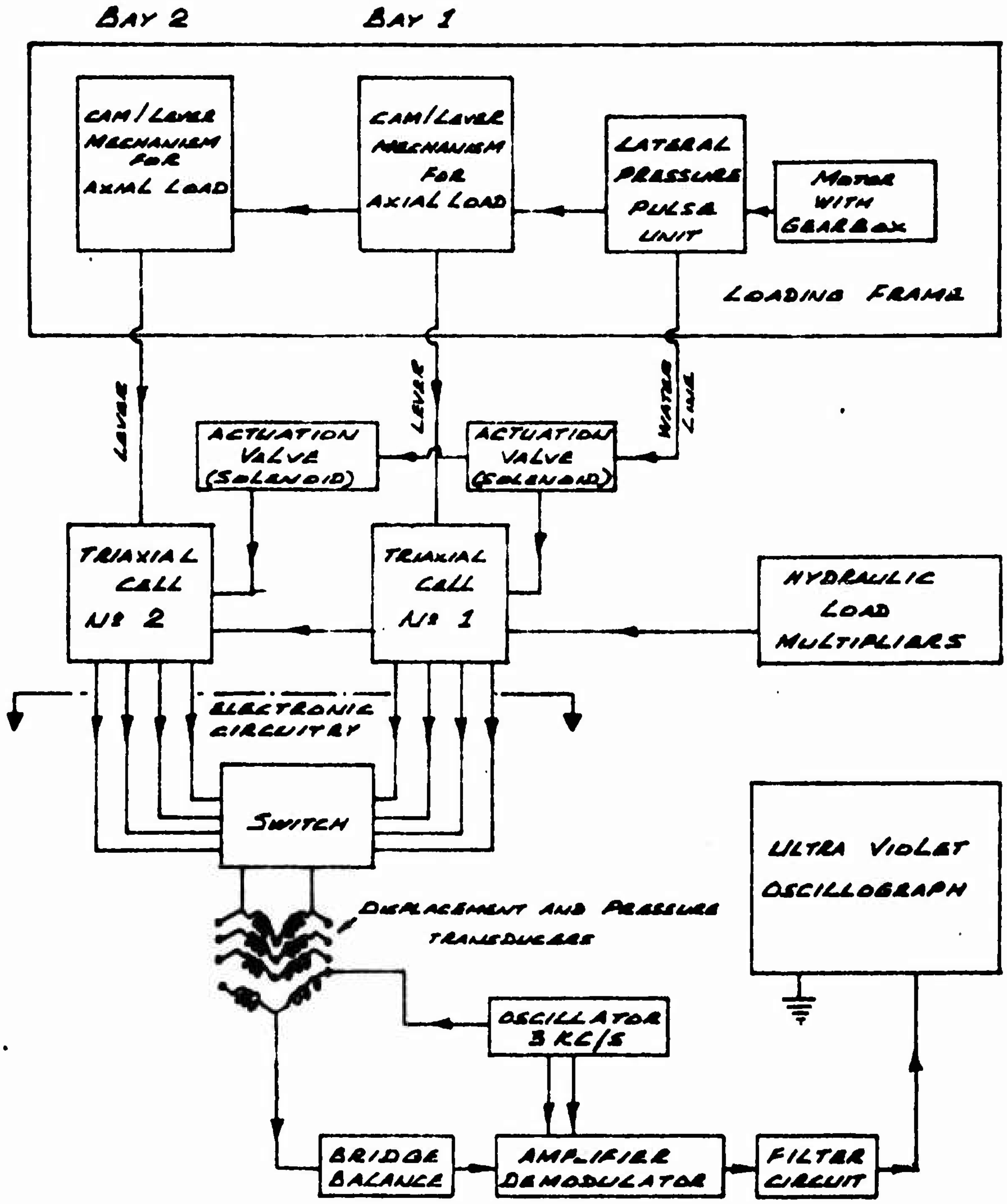
MOHR CIRCLES. FIG. 14



TYPICAL RESULTS OF SLOW UNDRAINED TRIAXIAL TEST

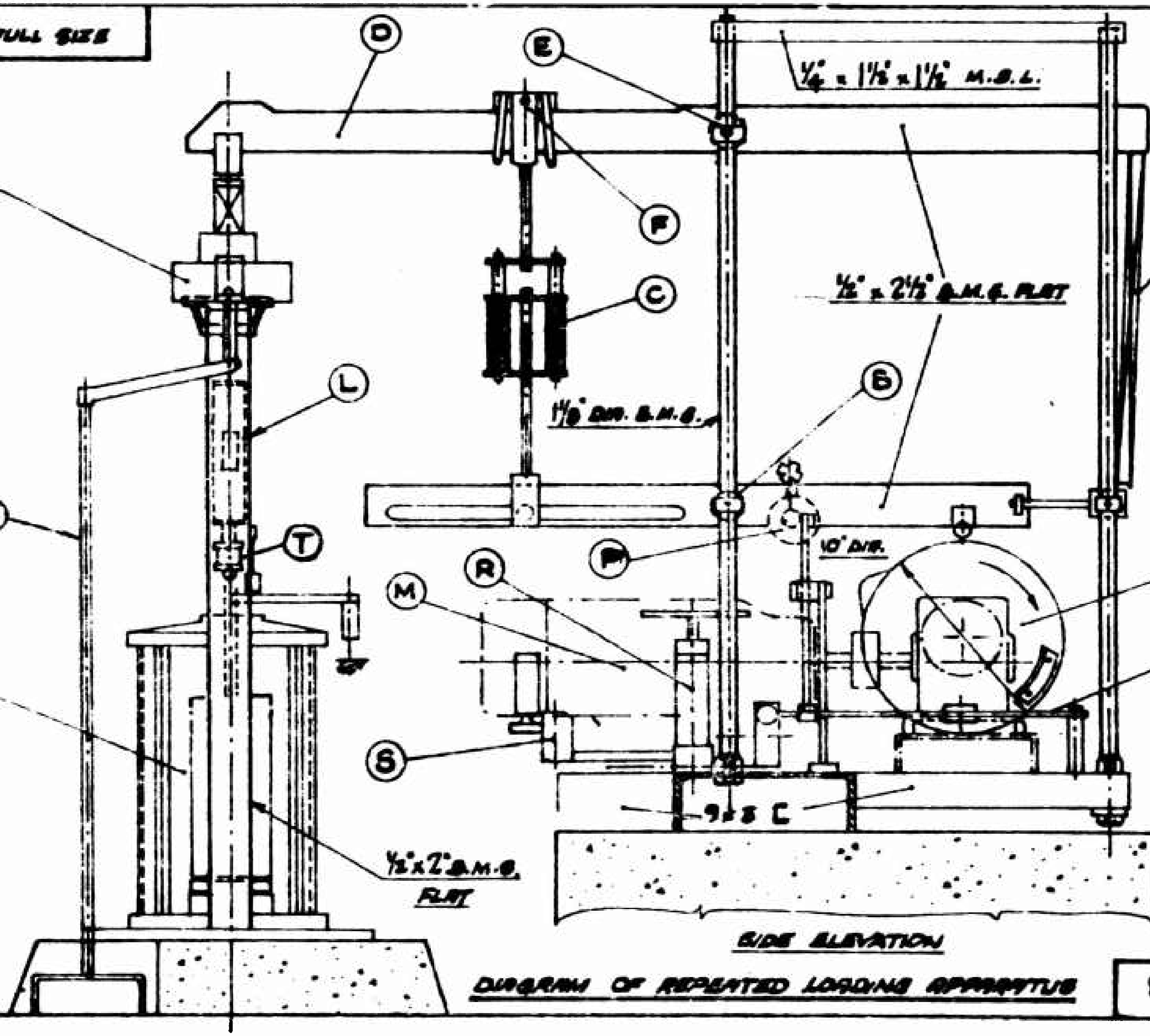
MOHR CIRCLES & VOLUME CHANGE OF SAMPLE vs. STRAIN

FIG. 15



BLOCK DIAGRAM OF REPEATED LOADING APPARATUS

SCALE:- 1/8 FULL SIZE



SIDE ELEVATION

DIAGRAM OF REPEATED LOADING APPARATUS

FIG. 17

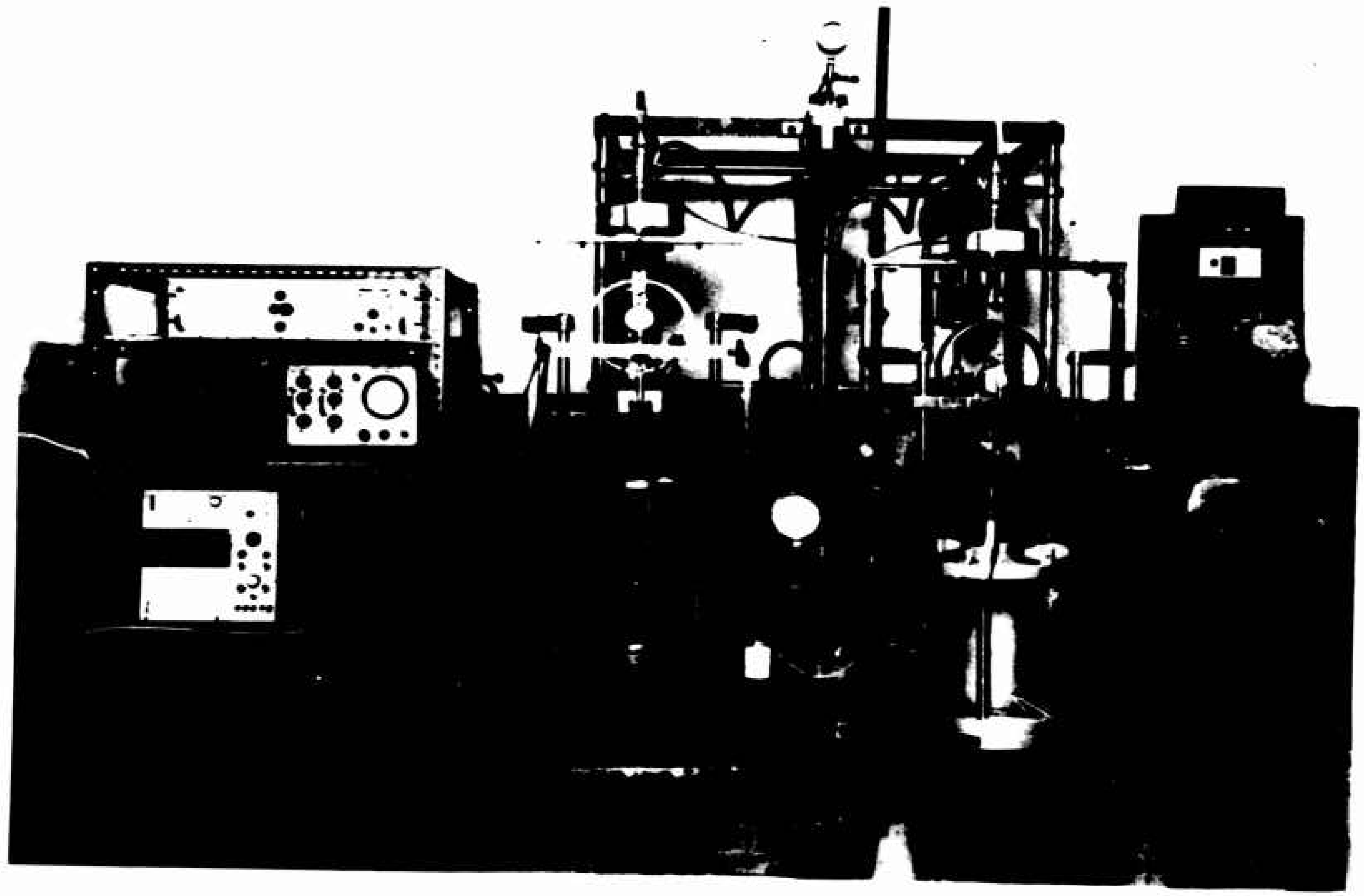
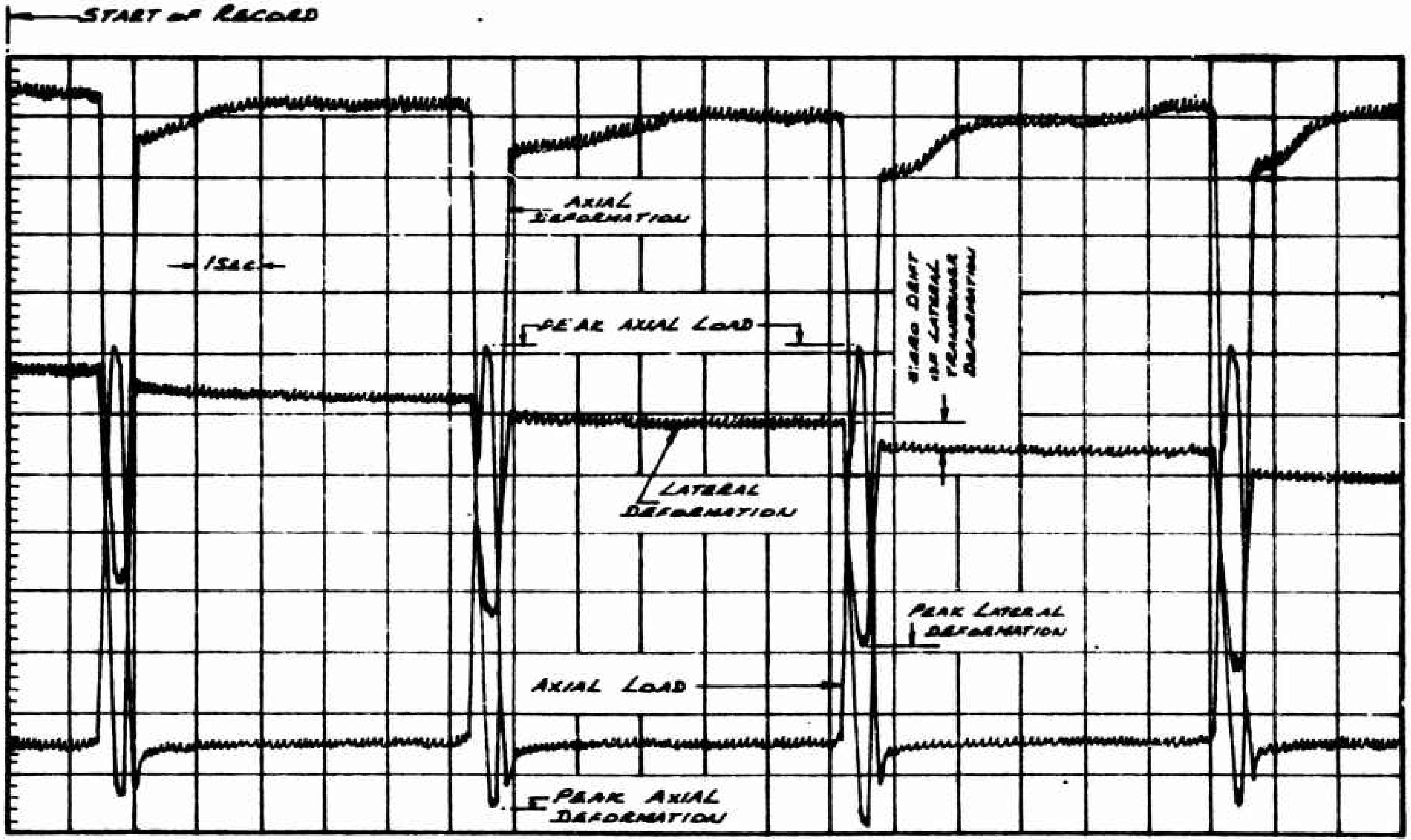
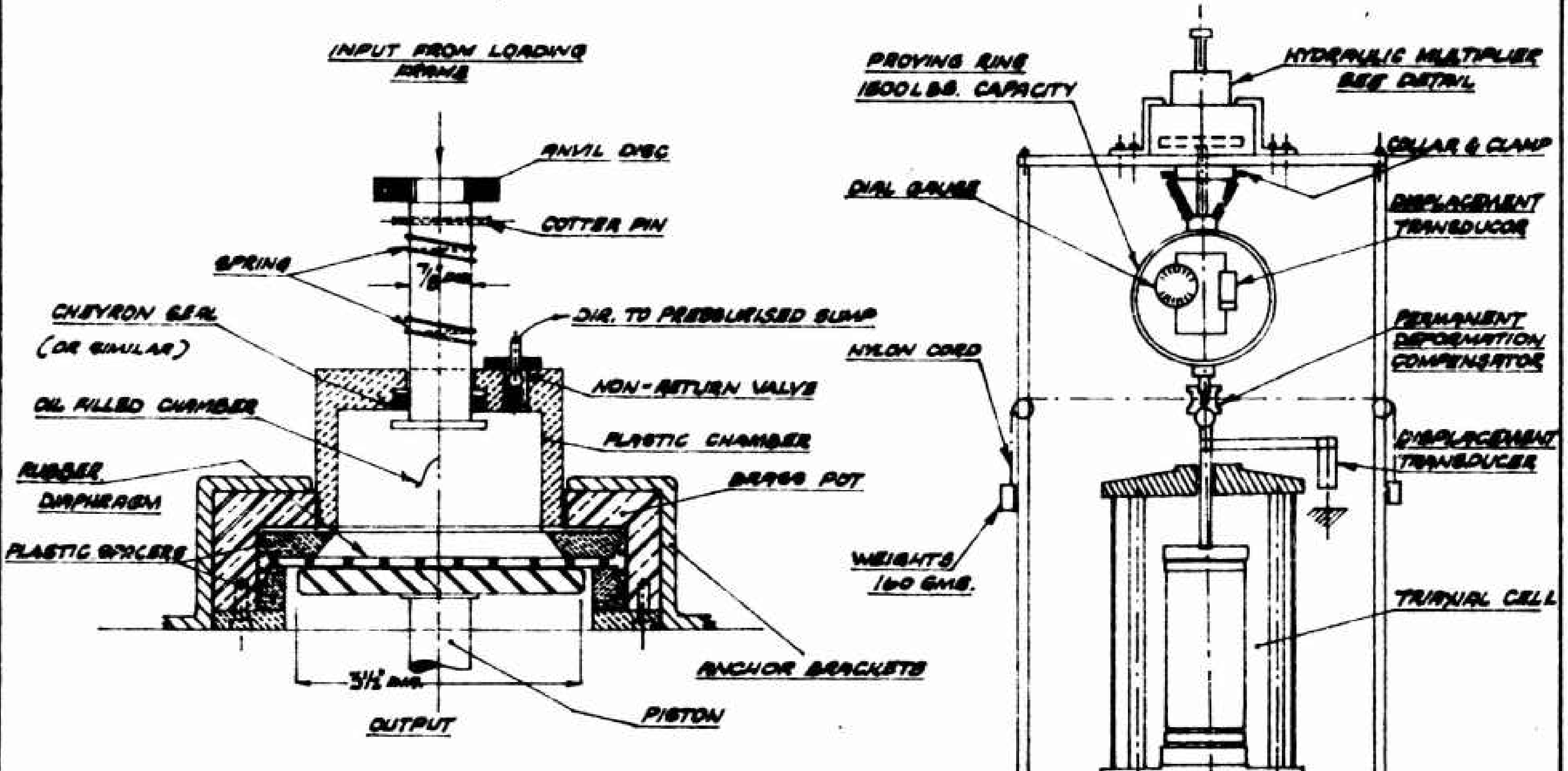


FIG 18 REPEATED LOADING MACHINE



TYPICAL TRACES OBTAINED DURING TESTS ON COMPACTED CLAY

FIG. 19

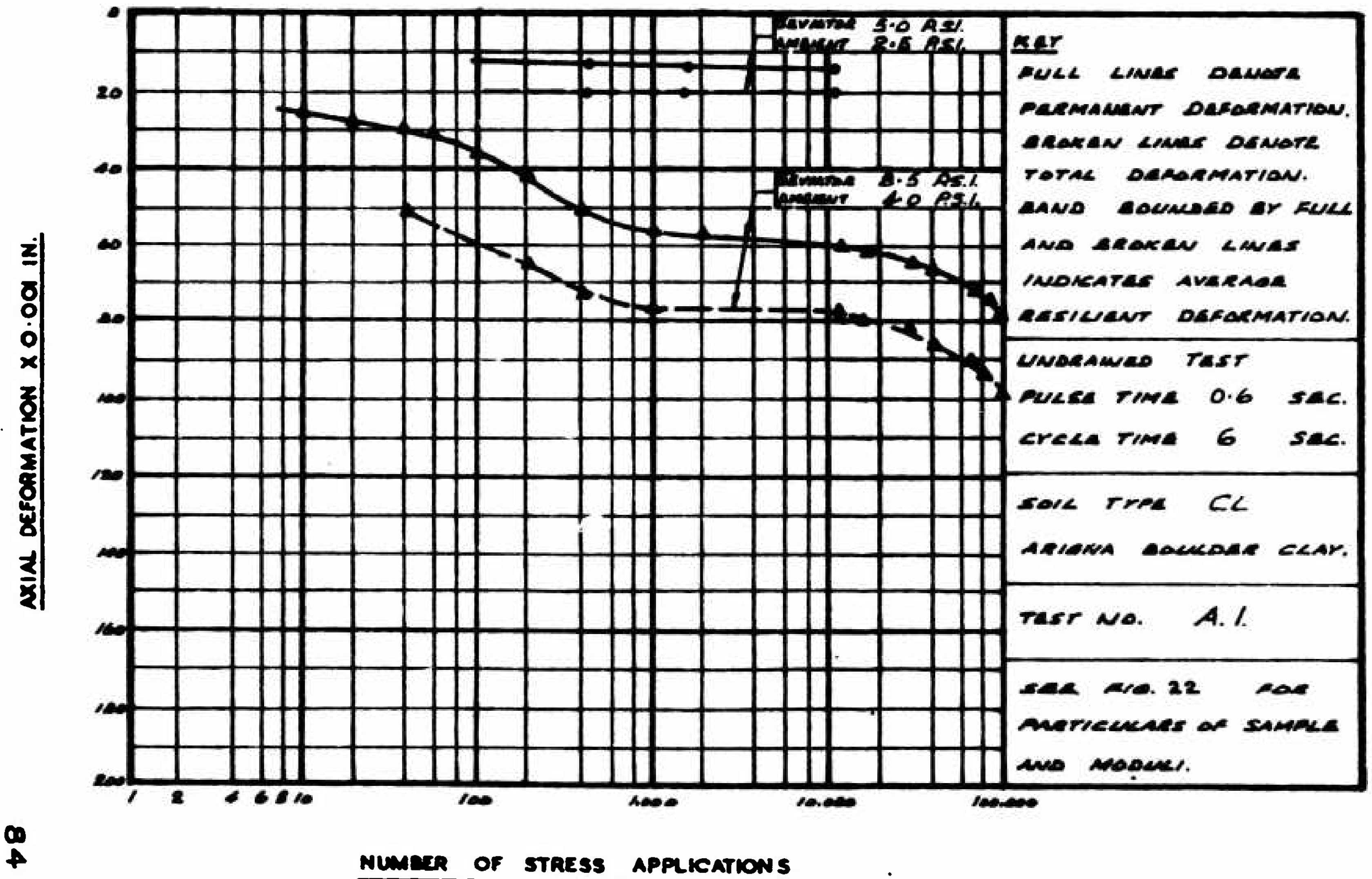


HYDRAULIC LOAD MULTIPLIER

(SCALE :- HALF FULL SIZE APPROX.)

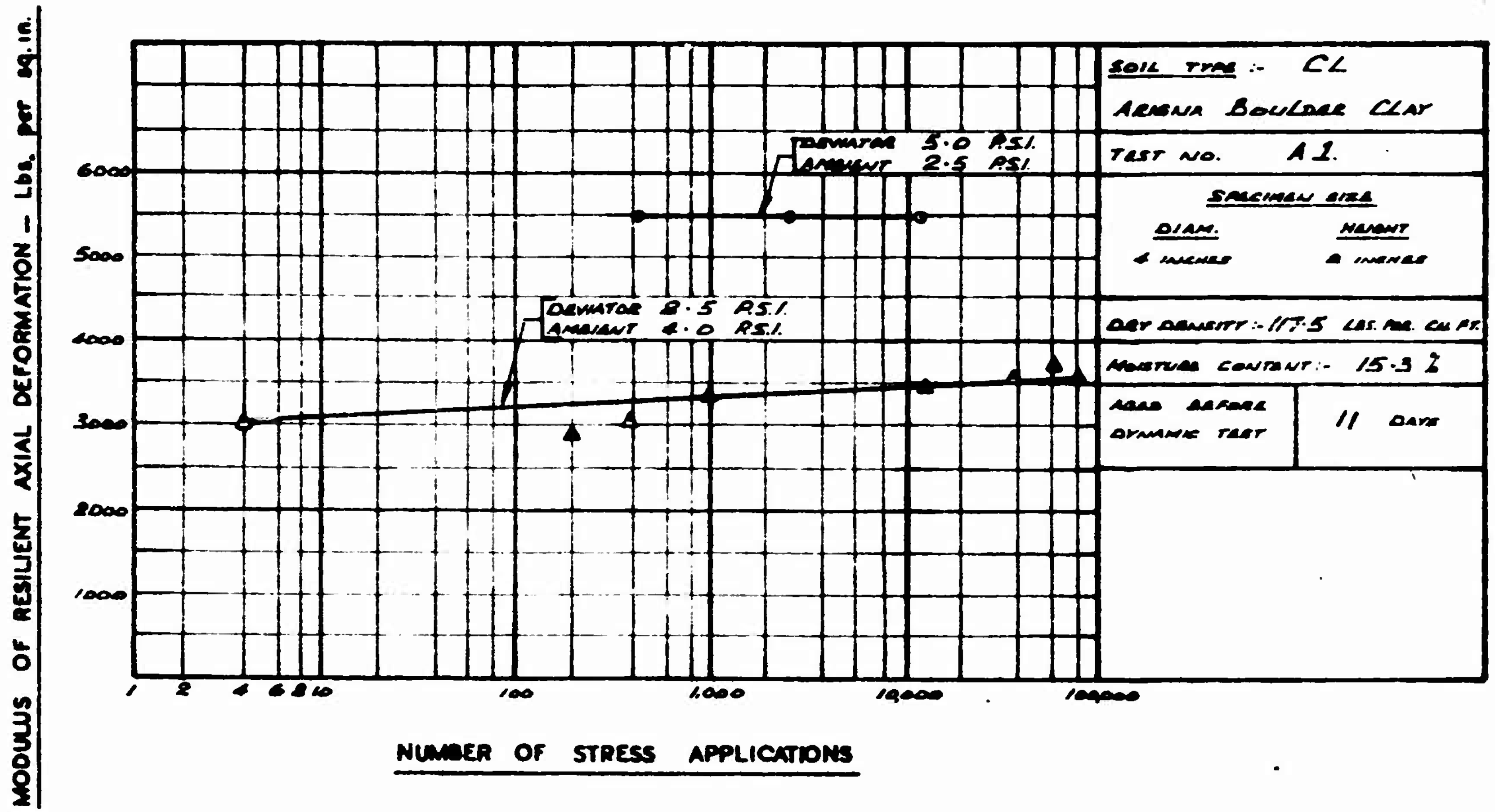
TRIAXIAL PRESS N.T.S.

FIG. 30



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

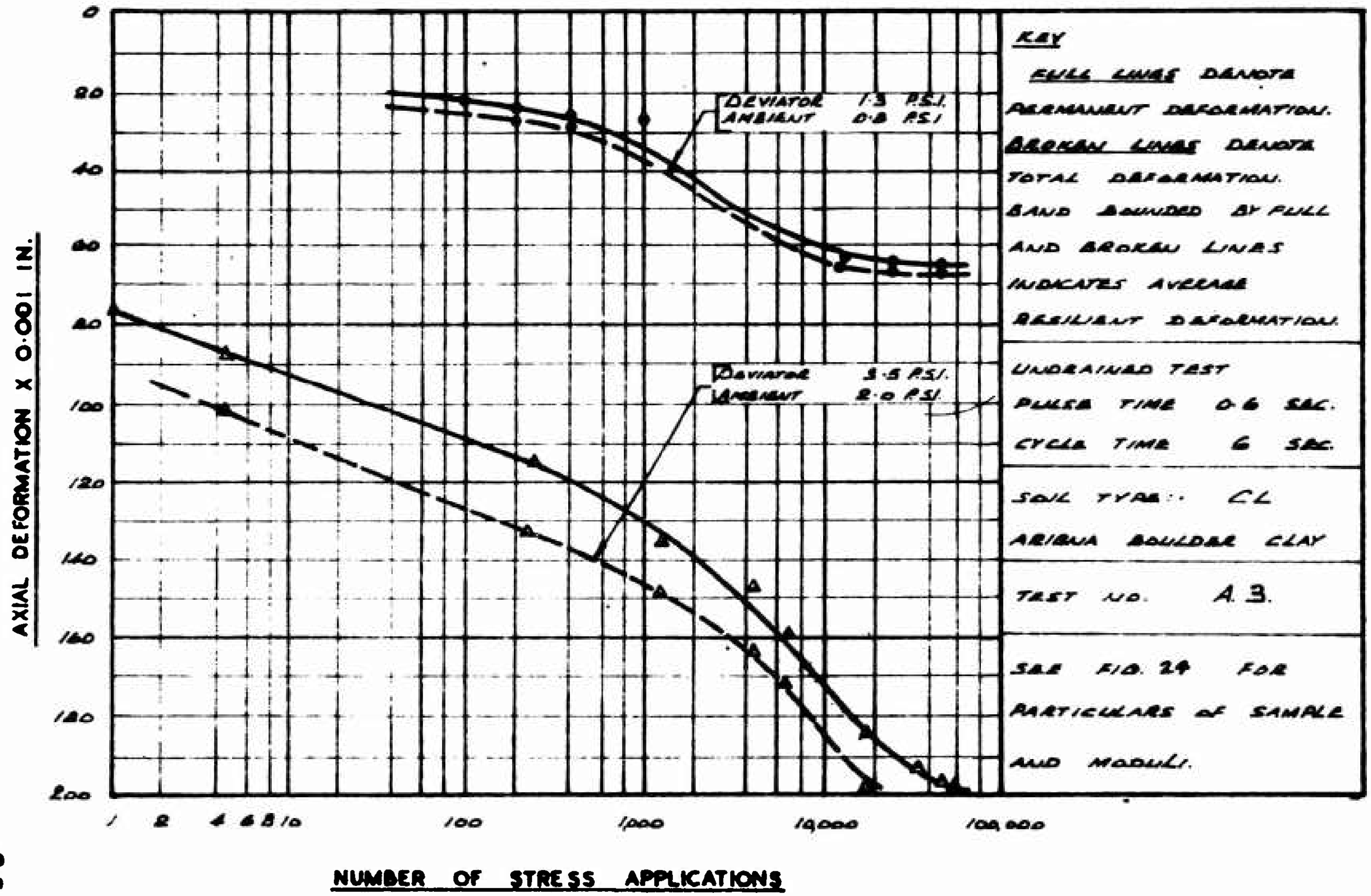
FIG. 21



DYNAMIC MODULUS — VS. — NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

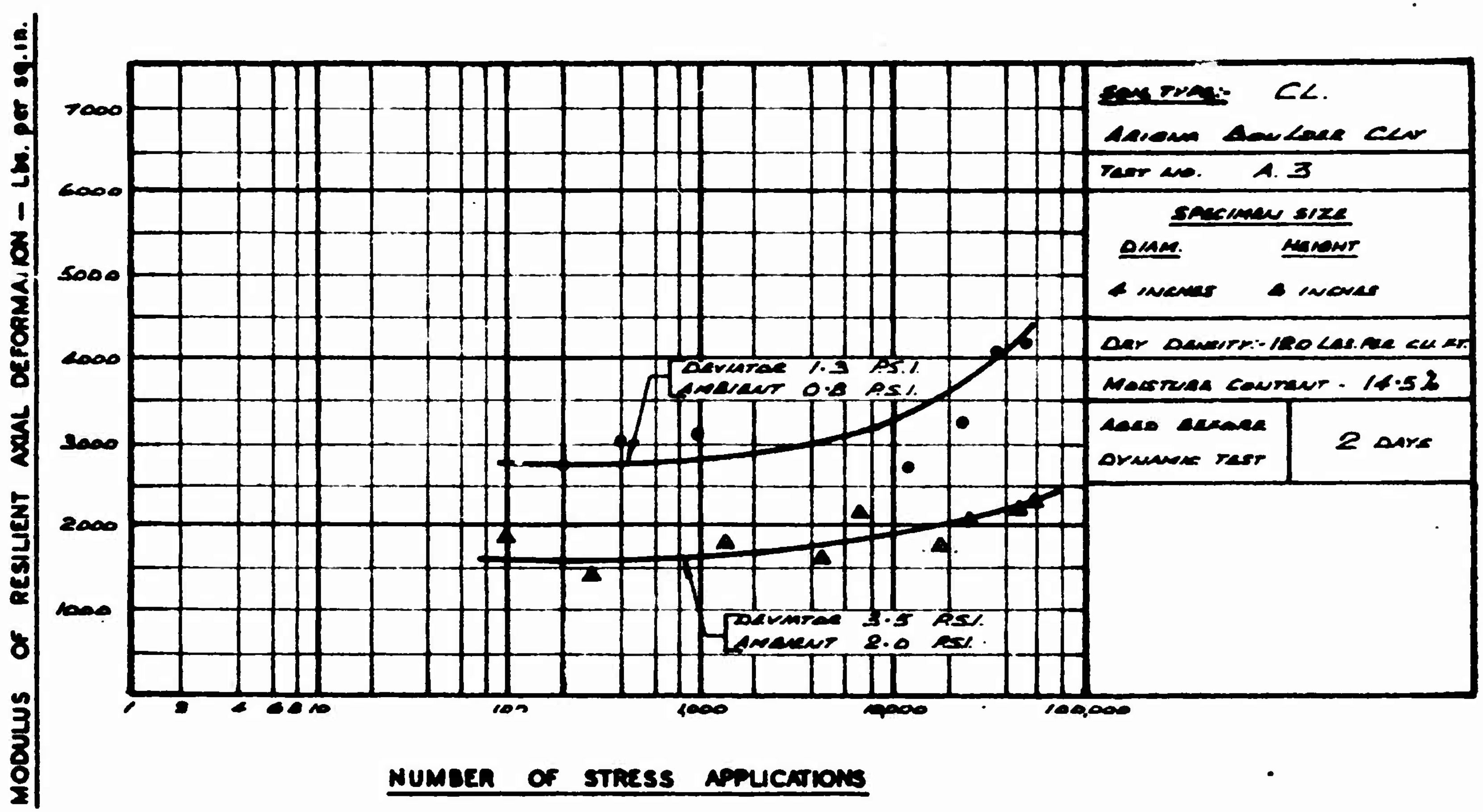
TRIAXIAL COMPRESSION TEST

FIG. 22



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

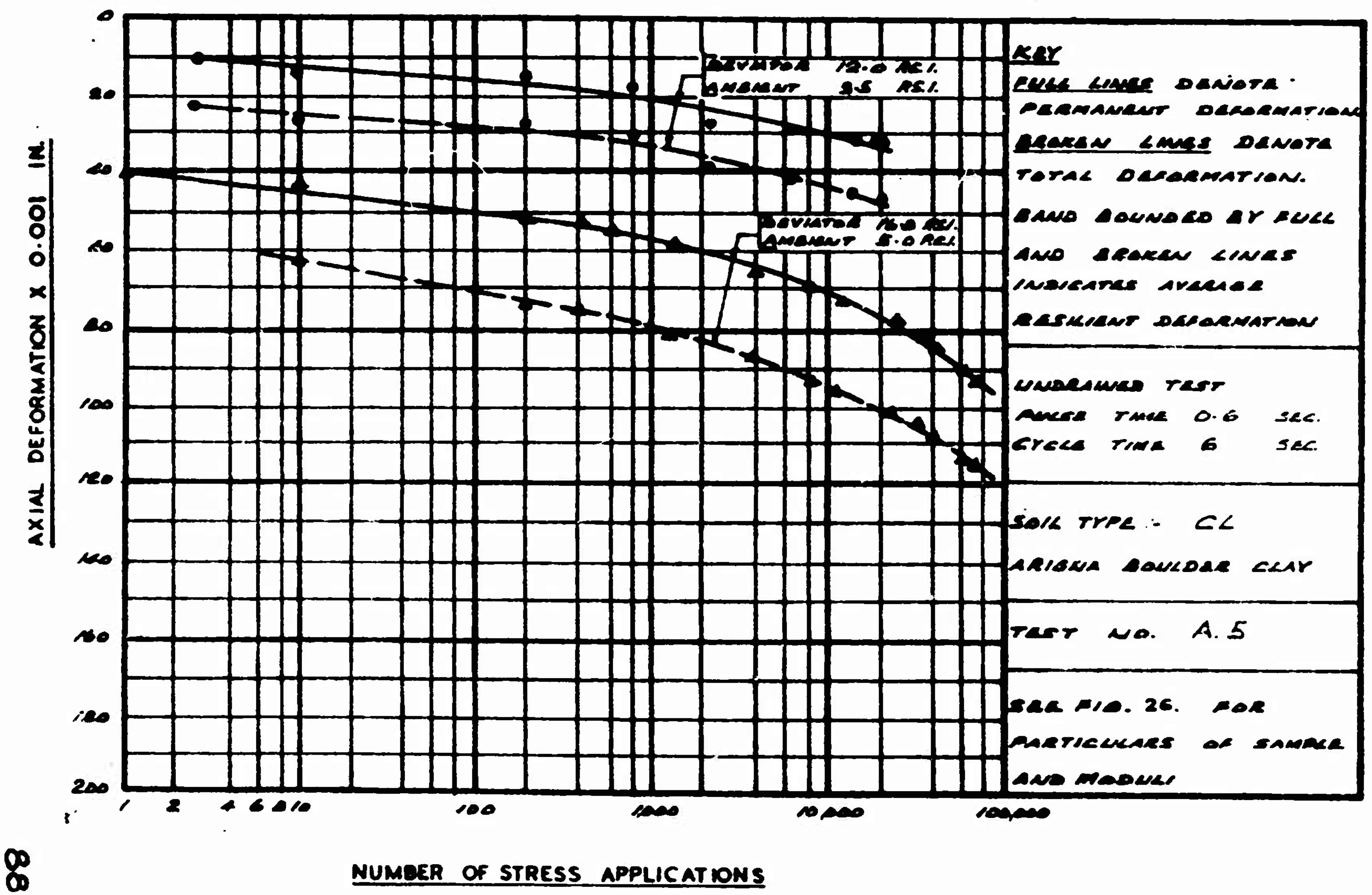
FIG. 23



DYNAMIC MODULUS — VS. — NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

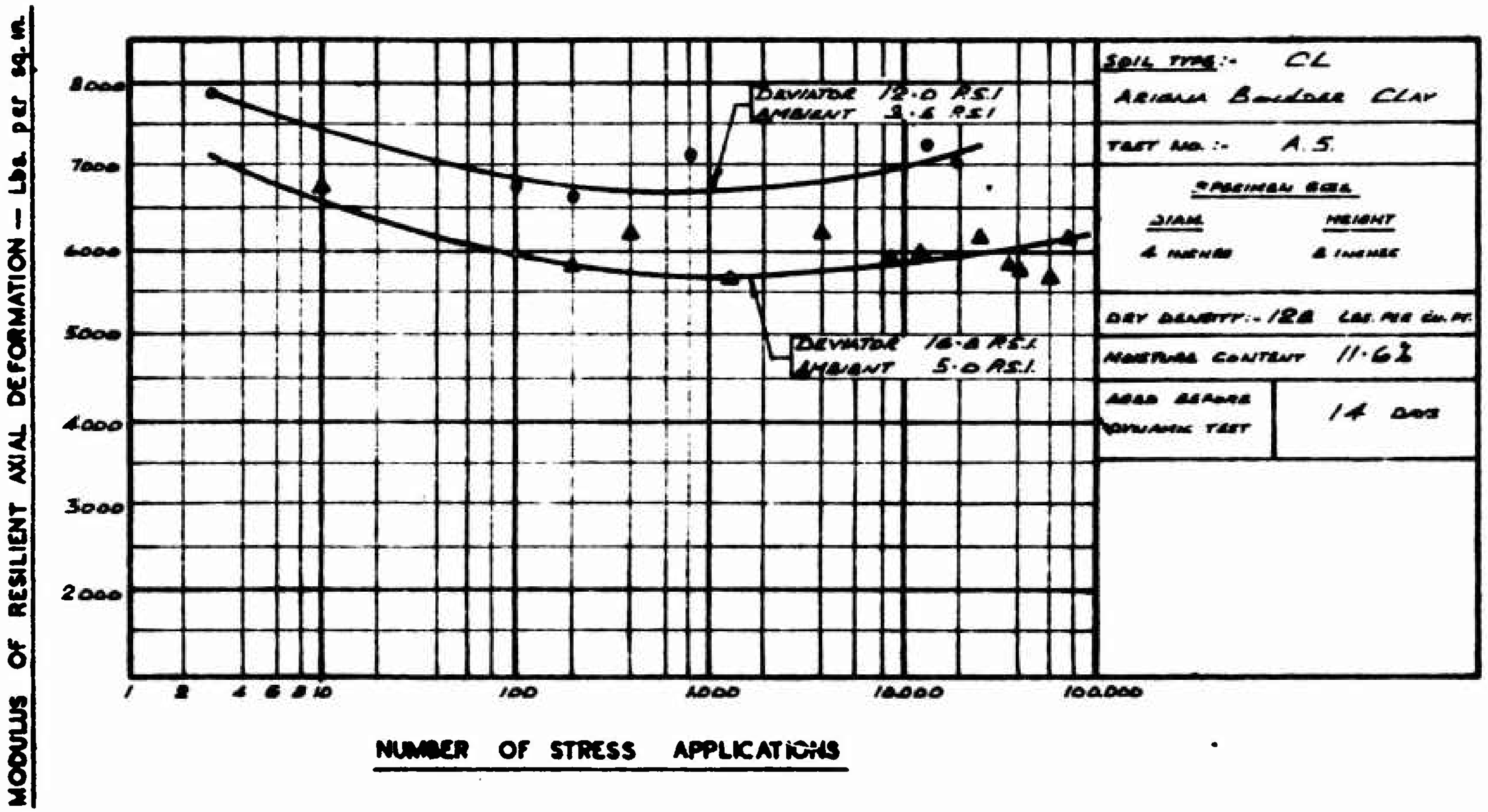
TRIAXIAL COMPRESSION TEST

FIG. 24



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

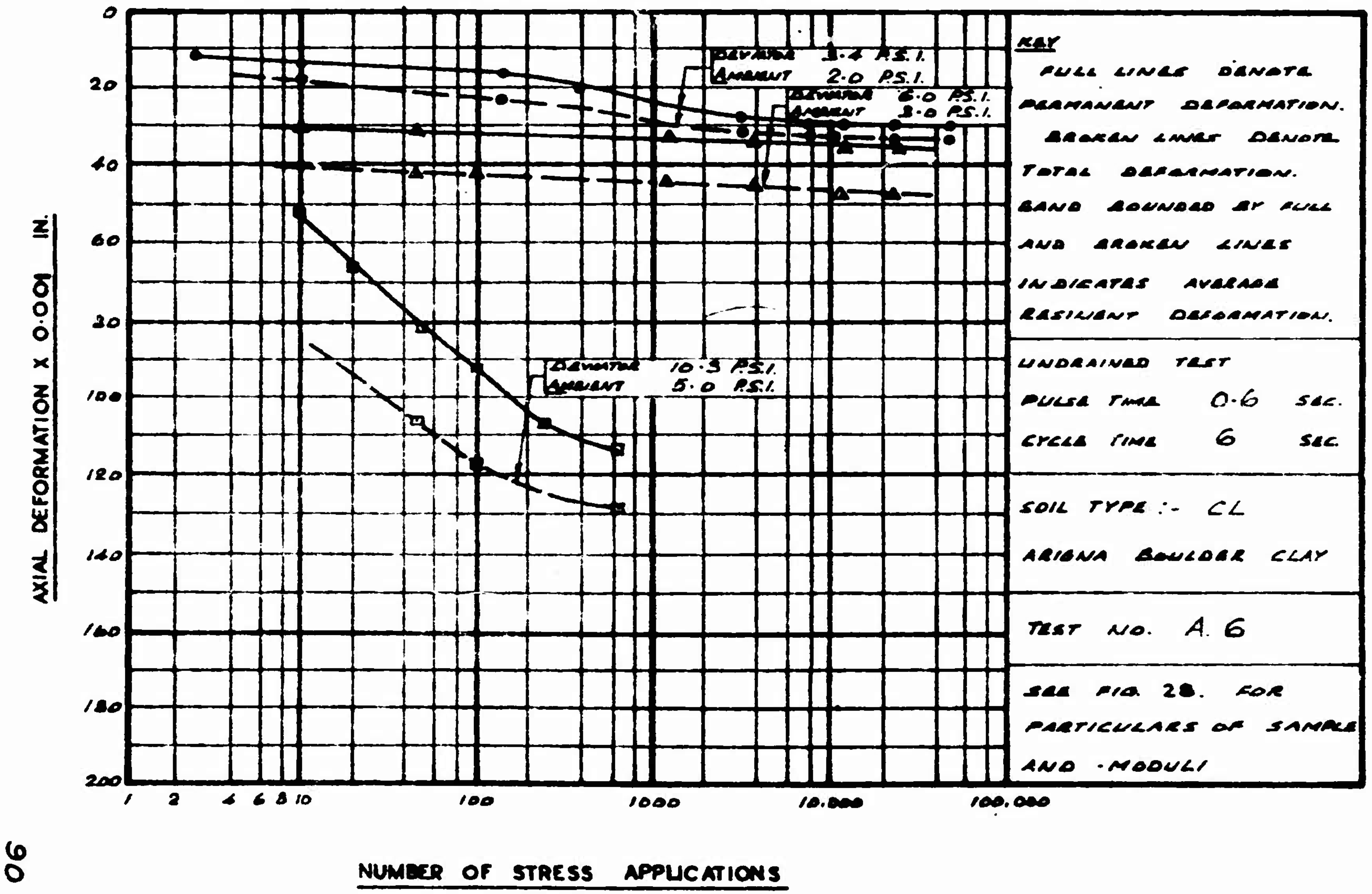
FIG. 25



DYNAMIC MODULUS – VS.– NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

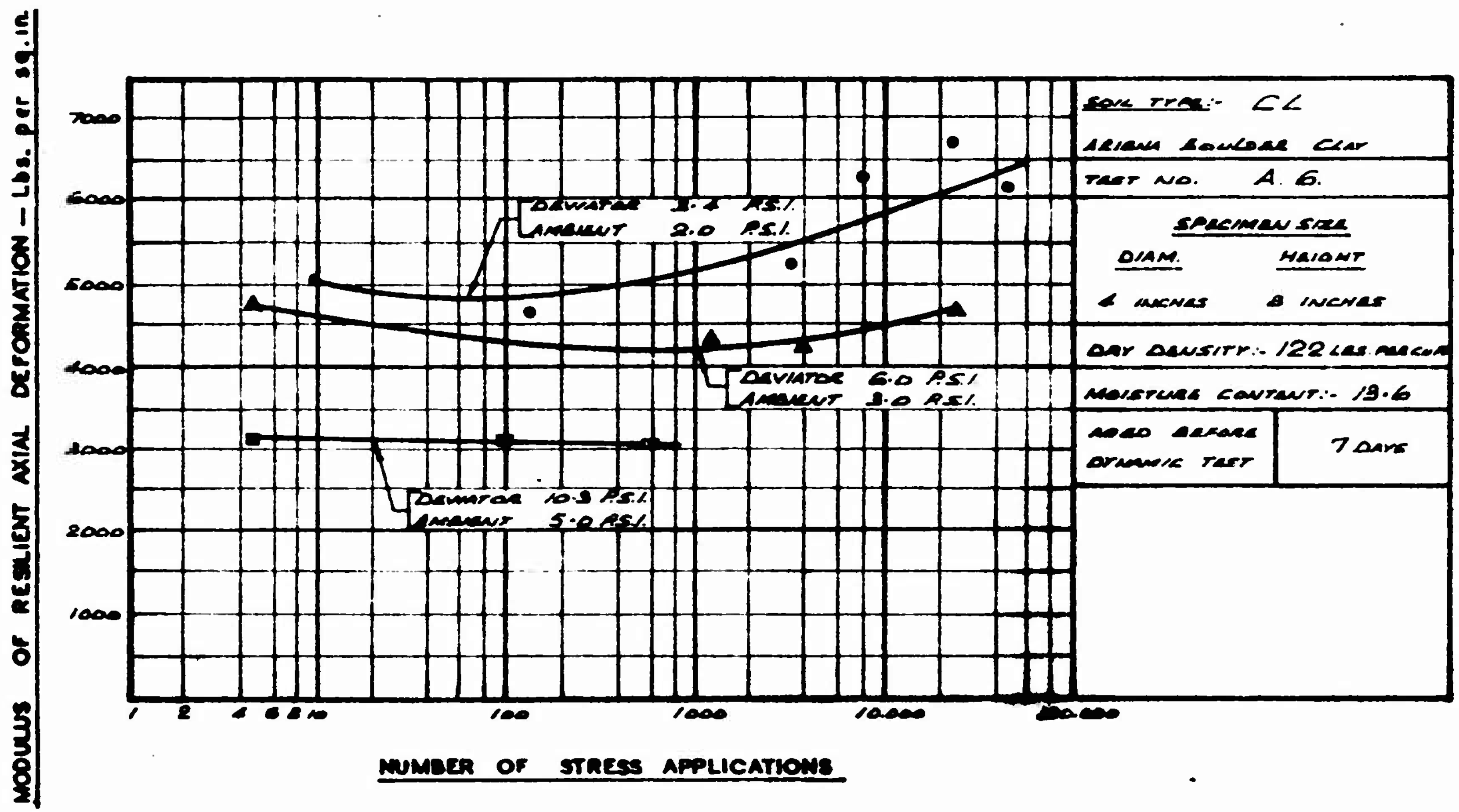
TRIAXIAL COMPRESSION TEST

FIG. 26



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

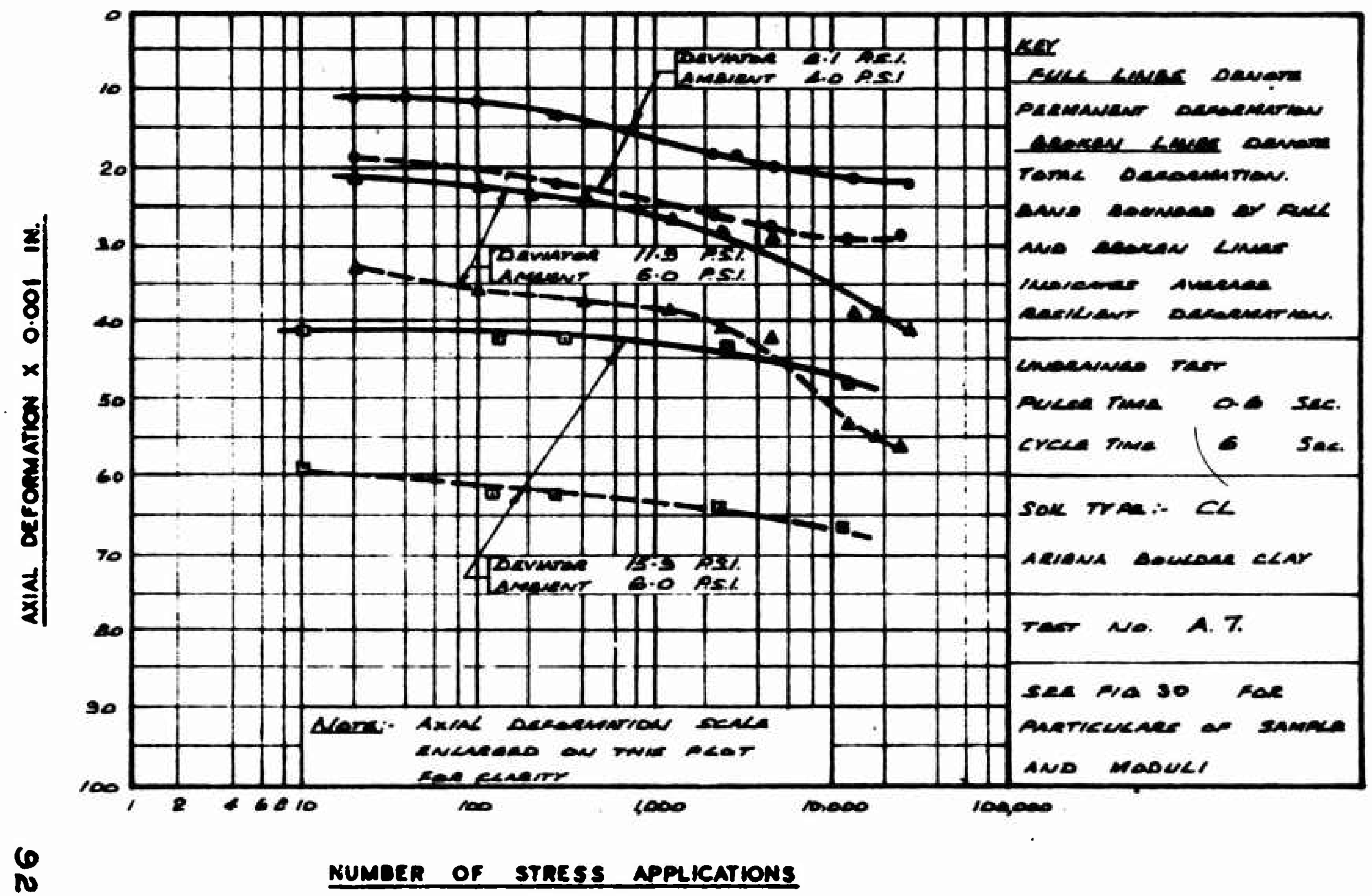
FIG. 27



DYNAMIC MODULUS – VS. – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

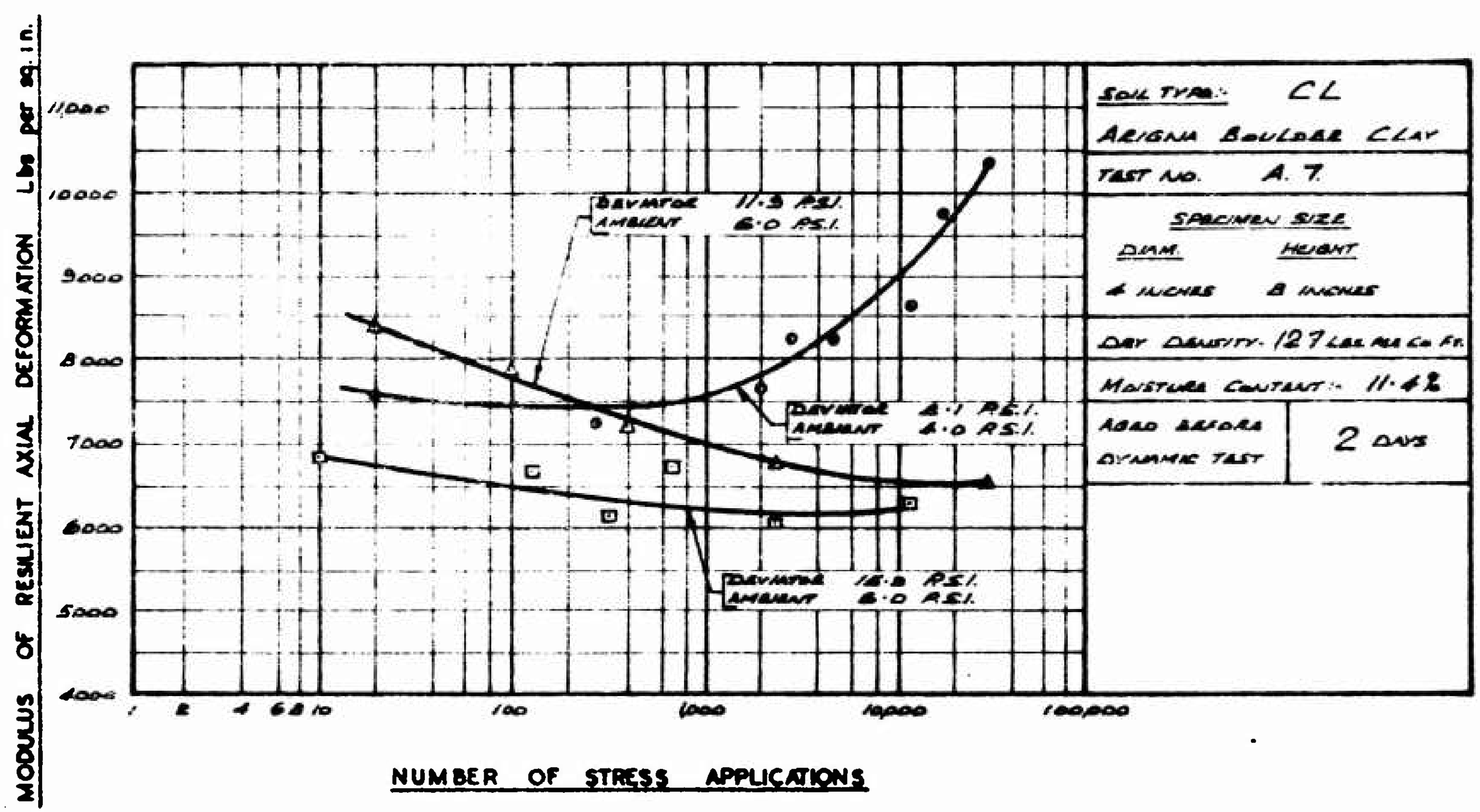
TRIAXIAL COMPRESSION TEST

FIG. 28



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

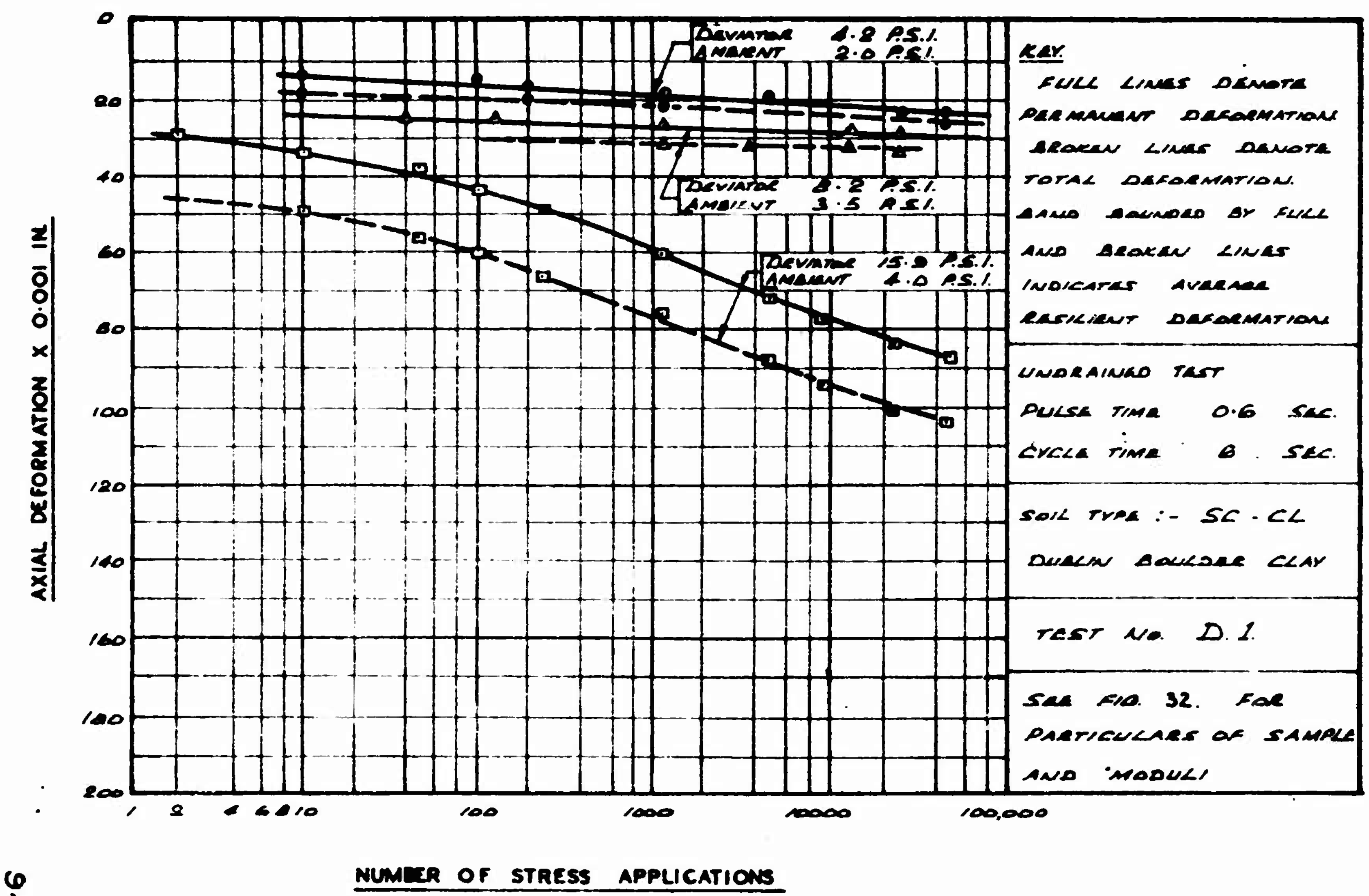
FIG. 29



DYNAMIC MODULUS - VS. - NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

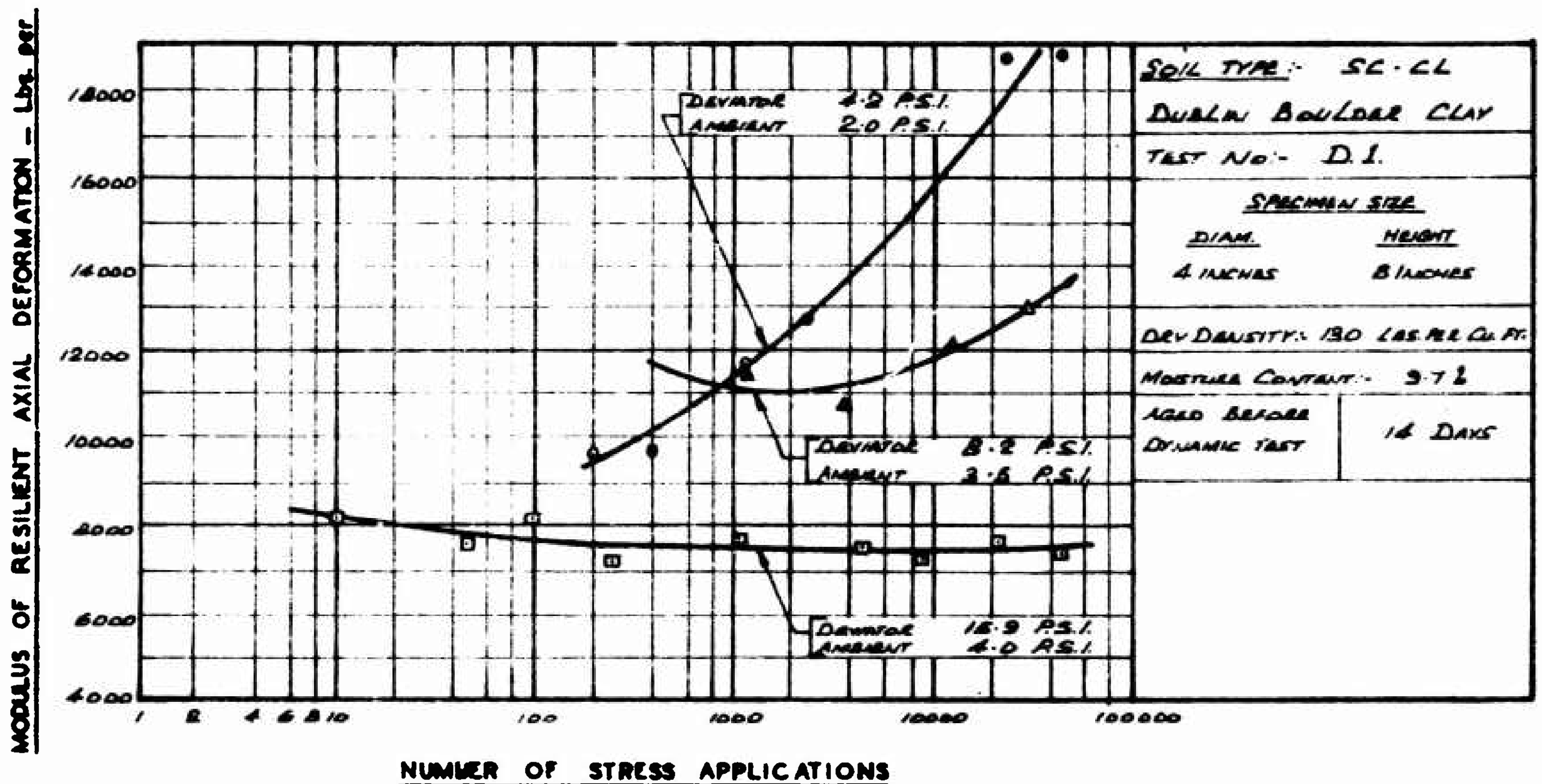
TRIAXIAL COMPRESSION TEST

FIG. 30



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

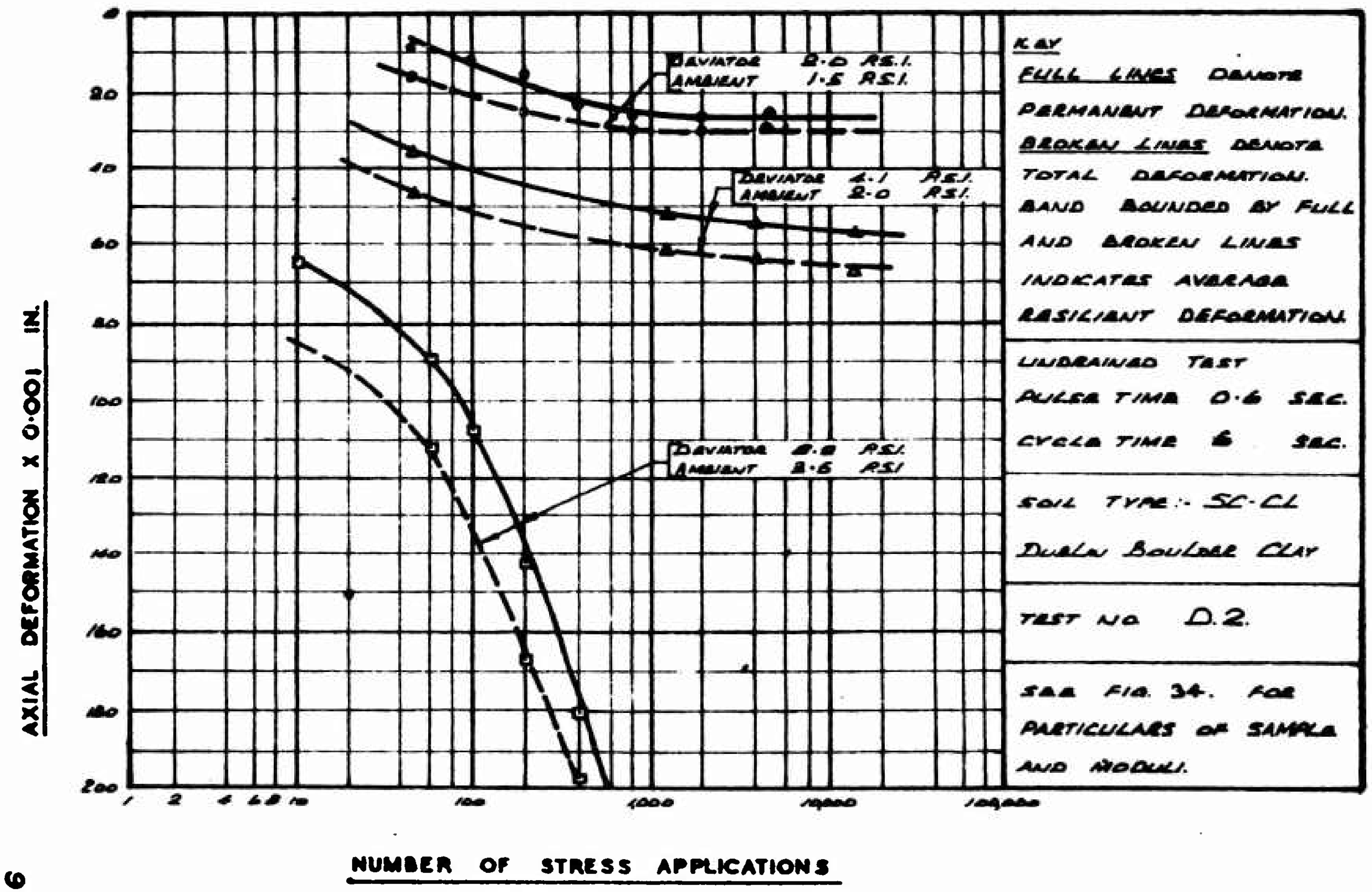
FIG. 31



DYNAMIC MODULUS – VS. – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

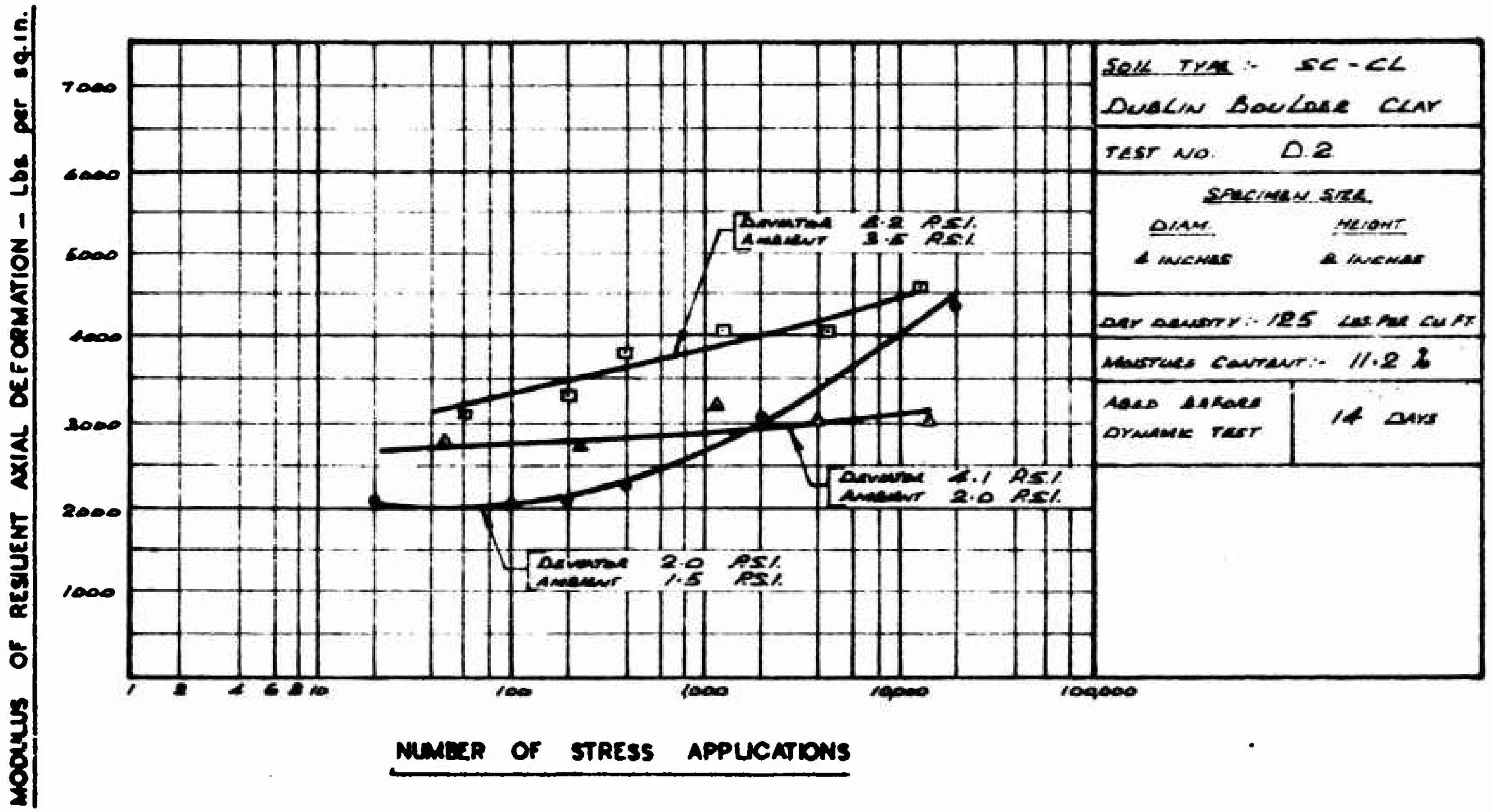
TRIAXIAL COMPRESSION TEST

FIG. 32



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

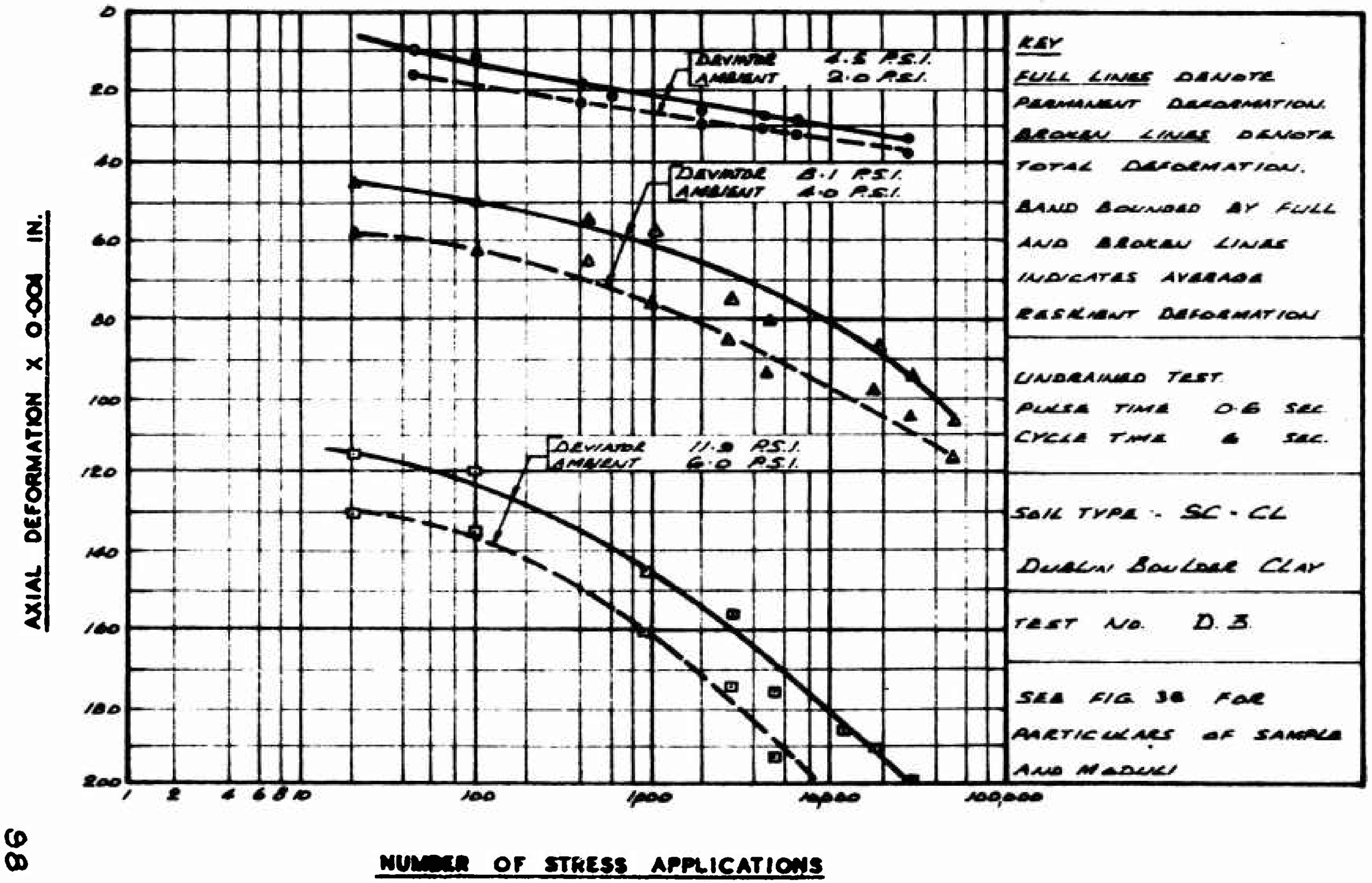
FIG. 33



DYNAMIC MODULUS – VS.– NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

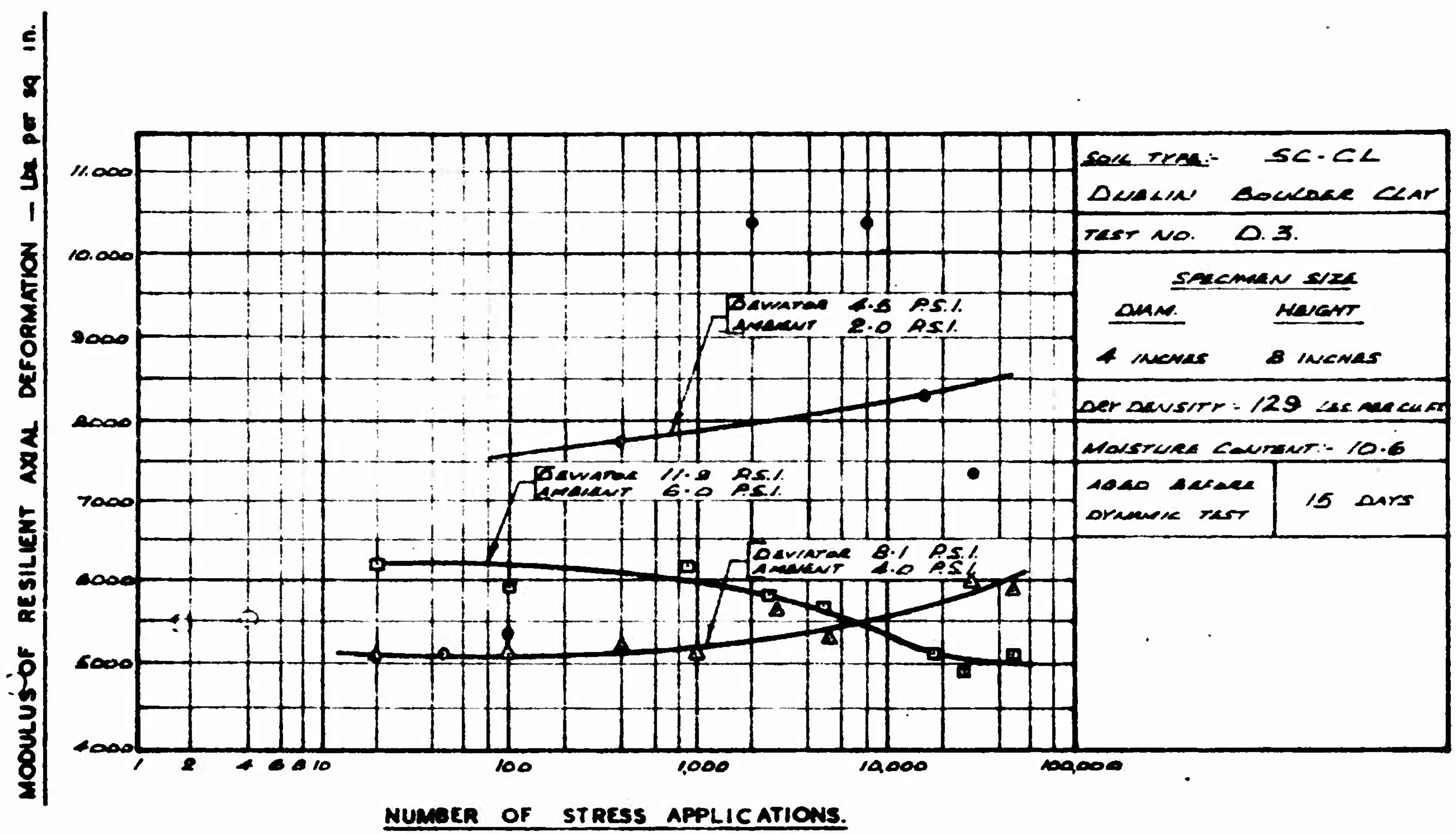
TRIAXIAL COMPRESSION TEST

FIG. 34



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

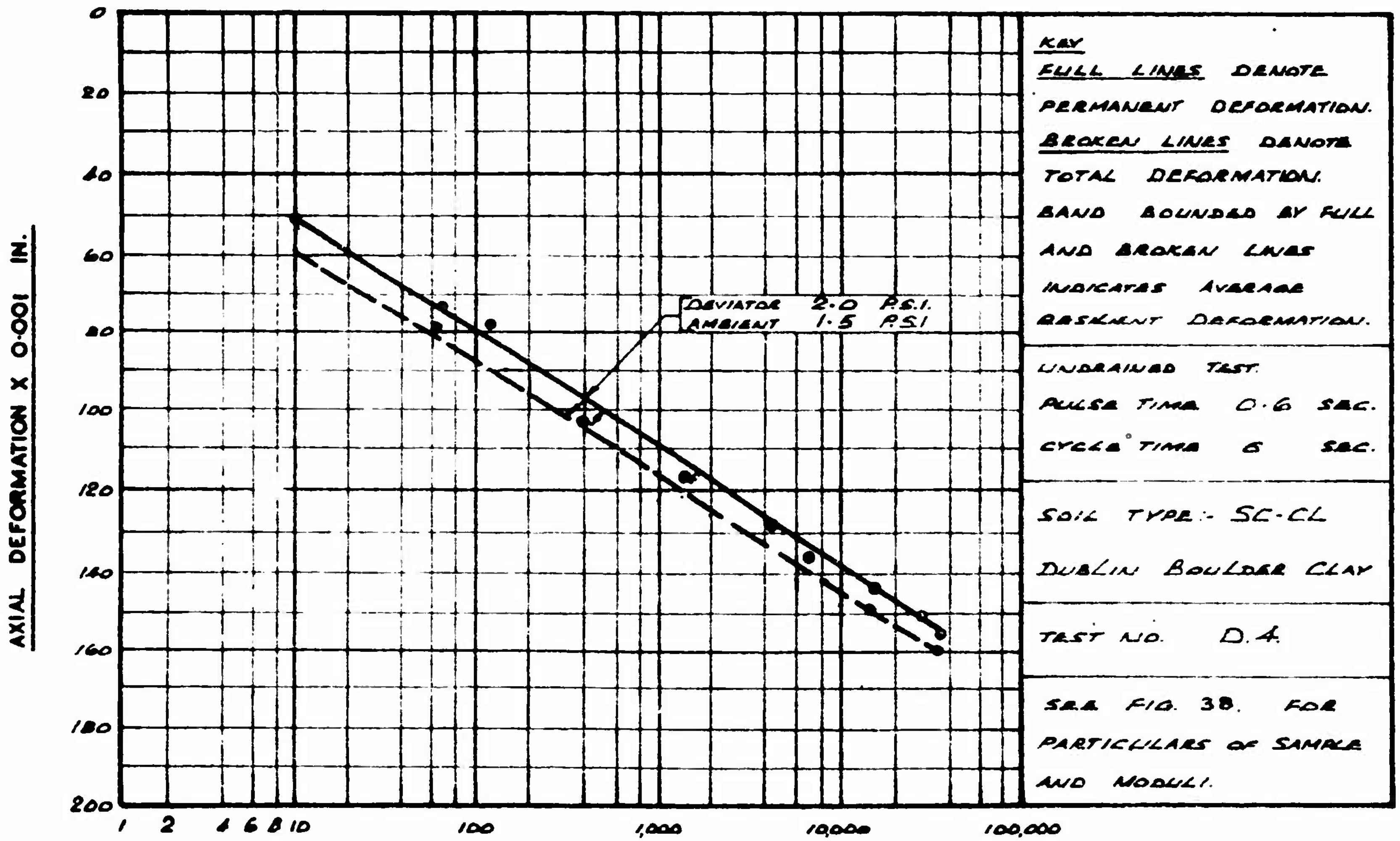
FIG. 35



DYNAMIC MODULUS - VS. - NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

TRIAXIAL COMPRESSION TEST

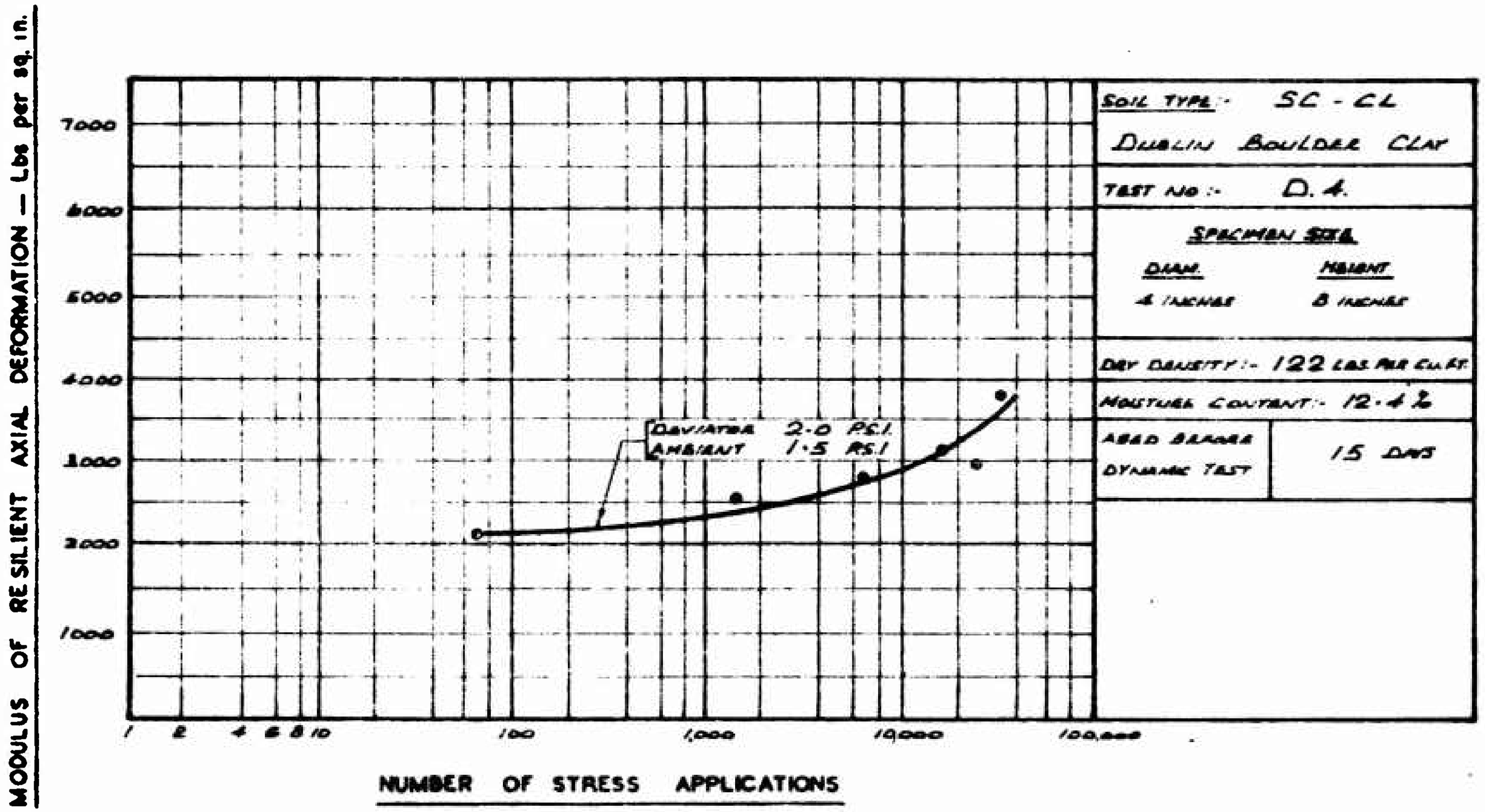
FIG. 36



NUMBER OF STRESS APPLICATIONS

RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

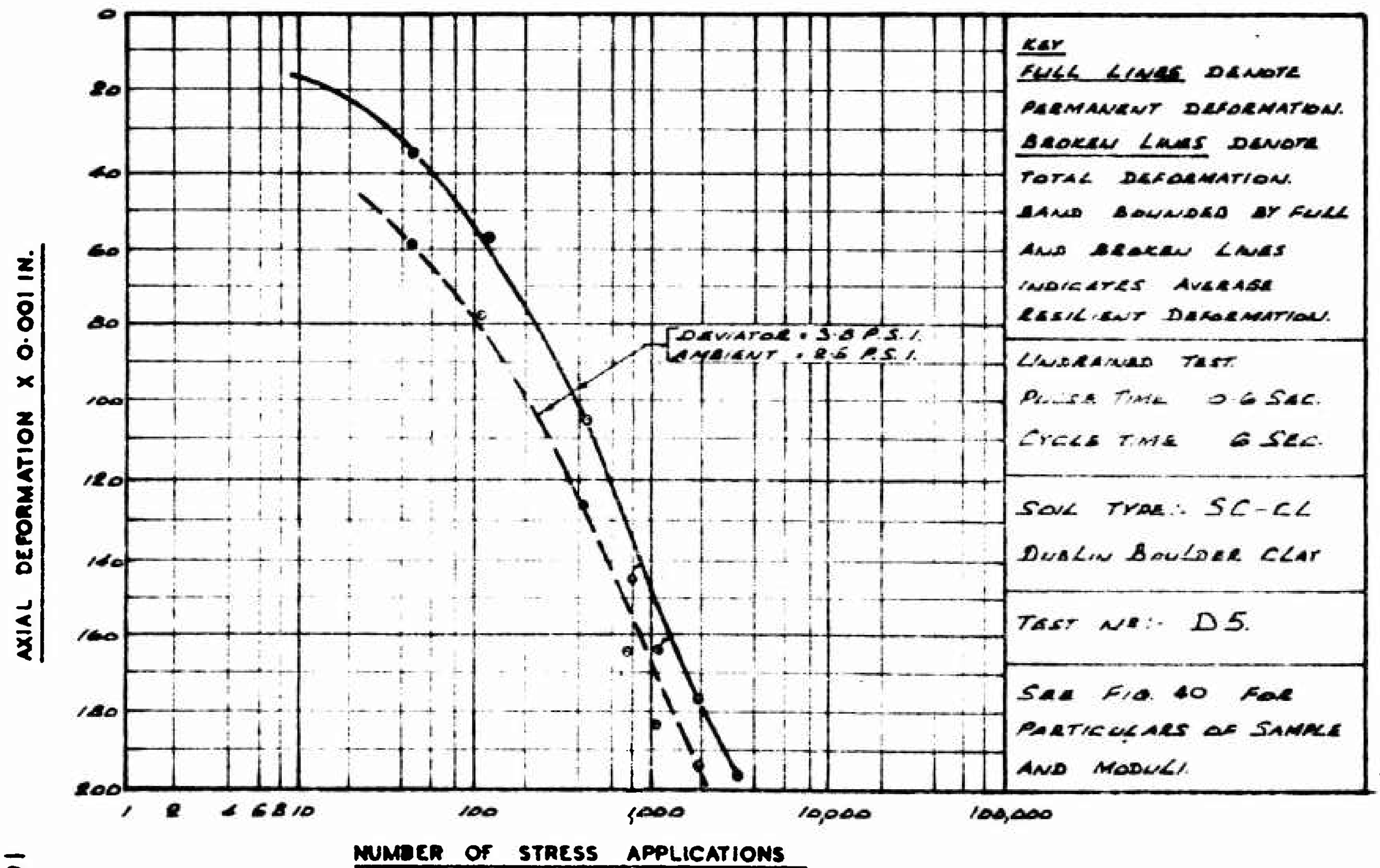
FIG. 37



DYNAMIC MODULUS - VS.- NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

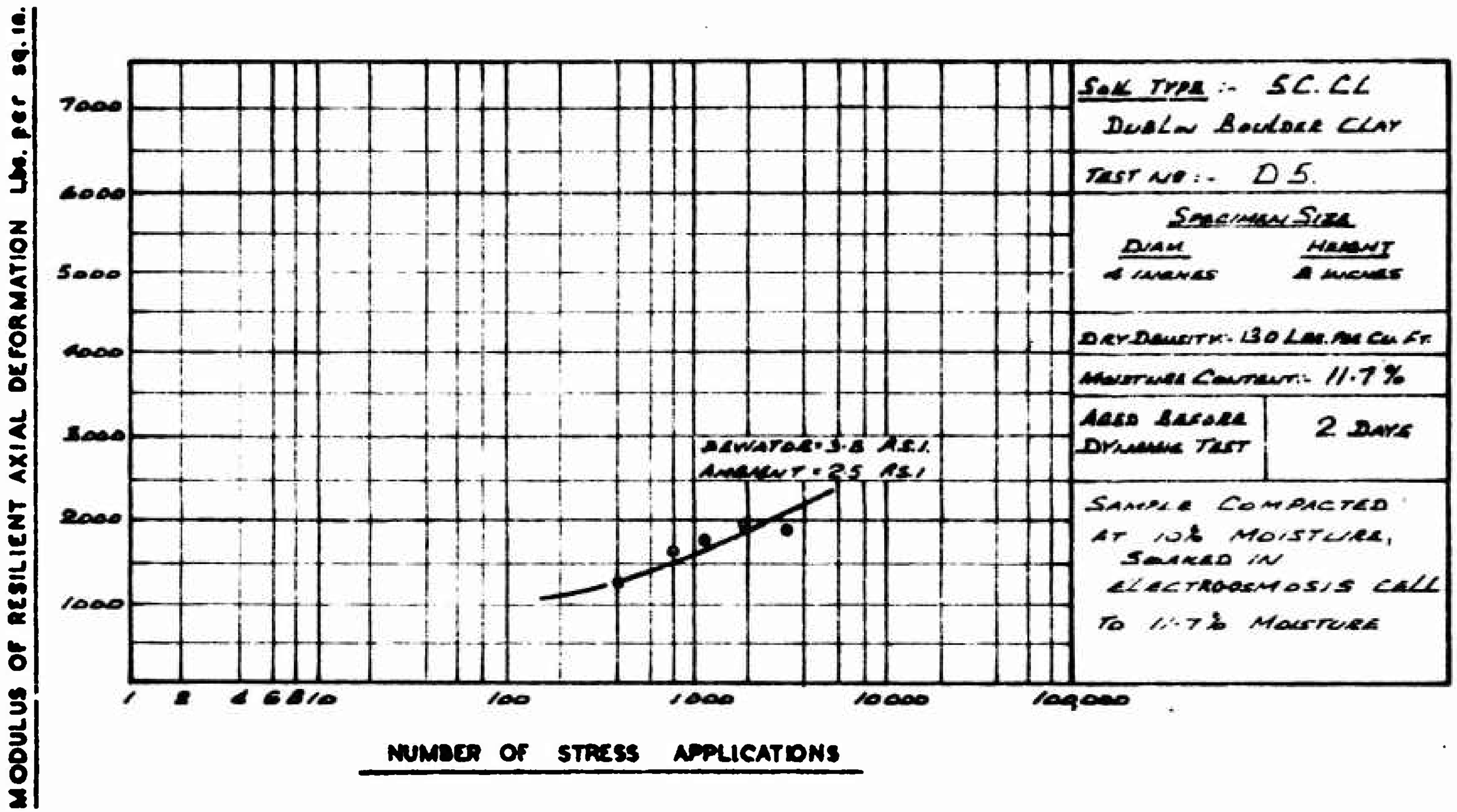
TRIAXIAL COMPRESSION TEST

FIG. 38



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

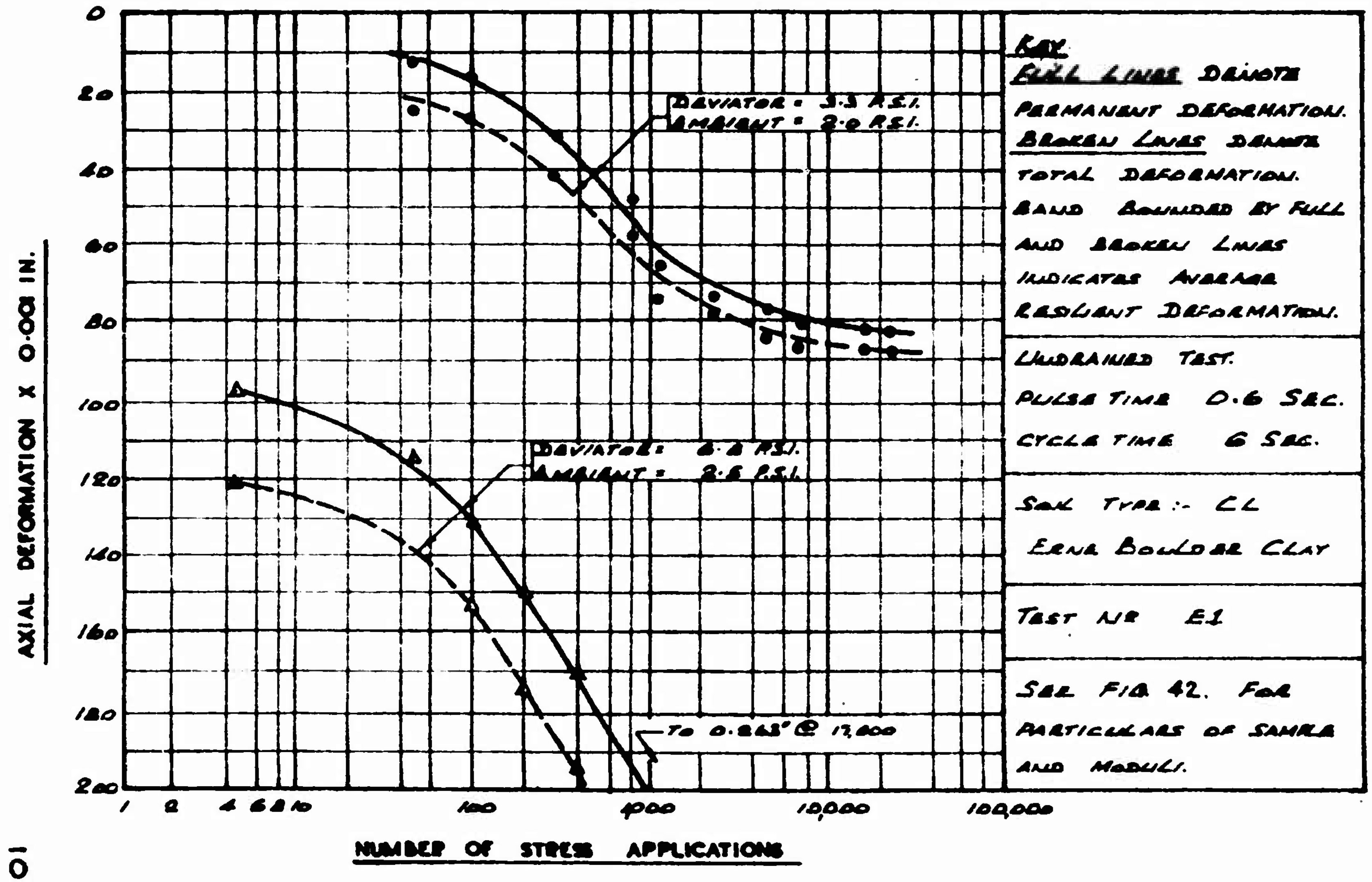
FIG.39



DYNAMIC MODULUS —VS.— NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

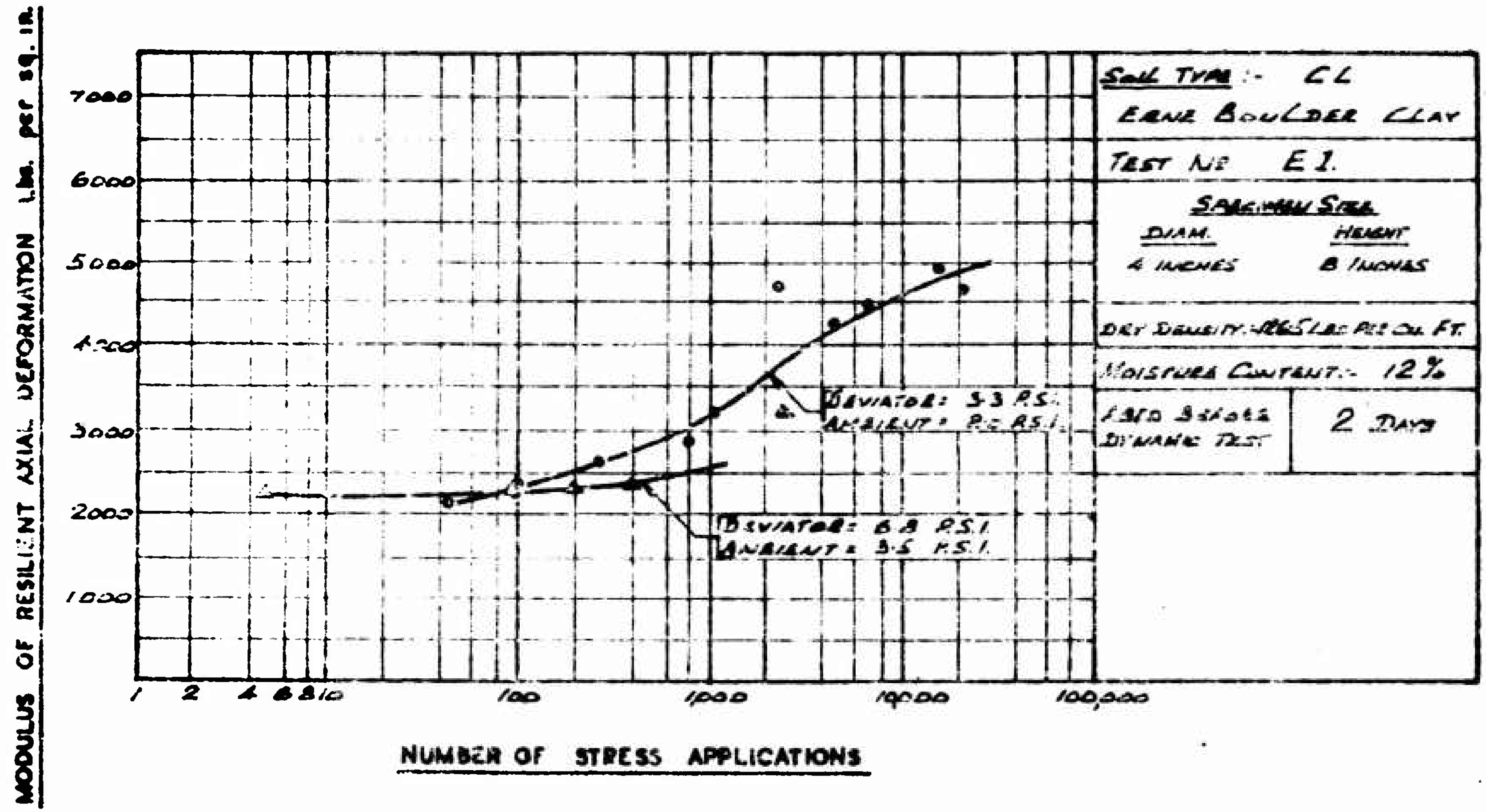
TRIAXIAL COMPRESSION TEST

FIG. 40



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

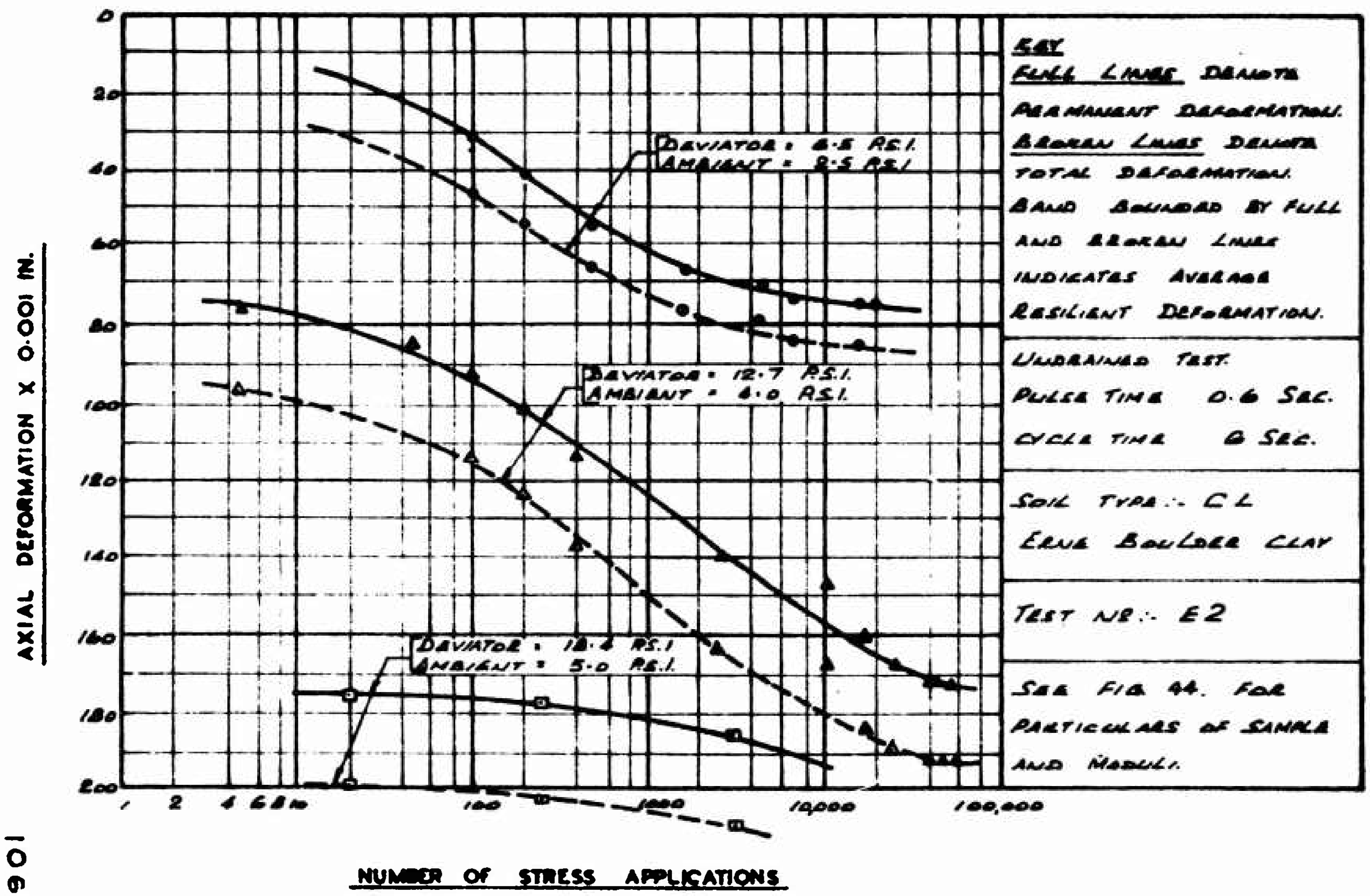
FIG. 41



DYNAMIC MODULUS — VS. — NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

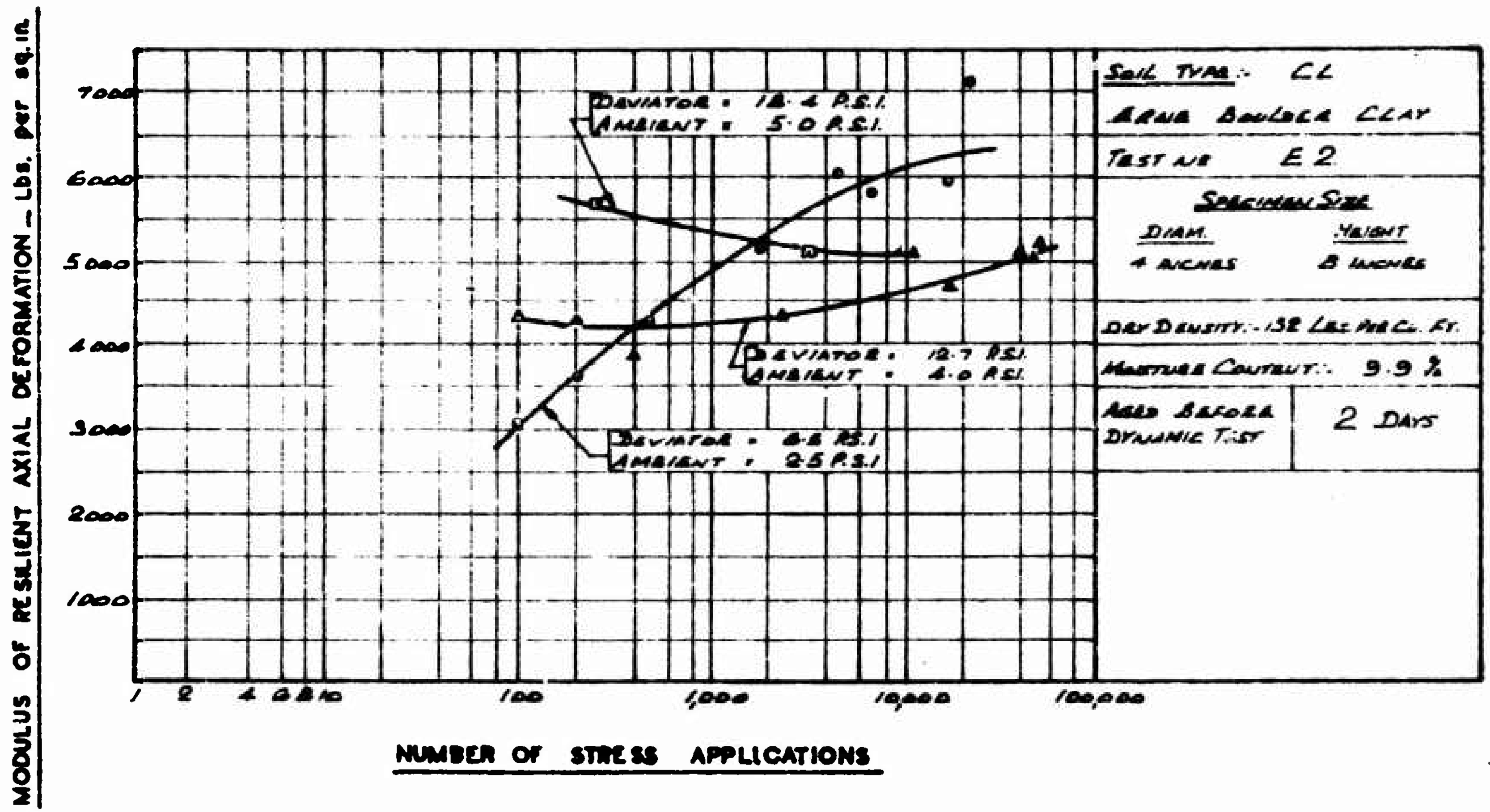
TRIAXIAL COMPRESSION TEST

FIG. 42



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

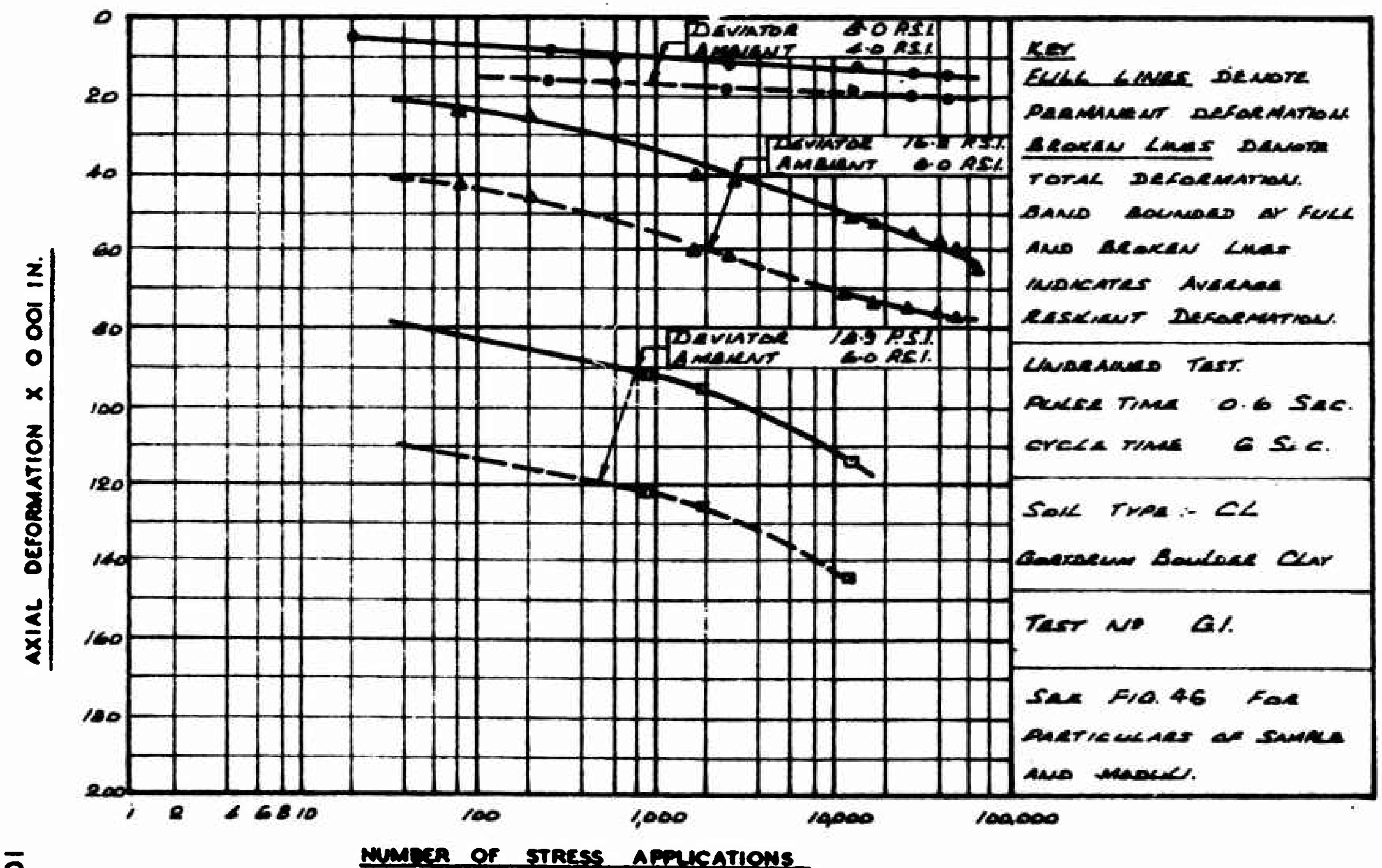
FIG. 43



DYNAMIC MODULUS —VS— NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

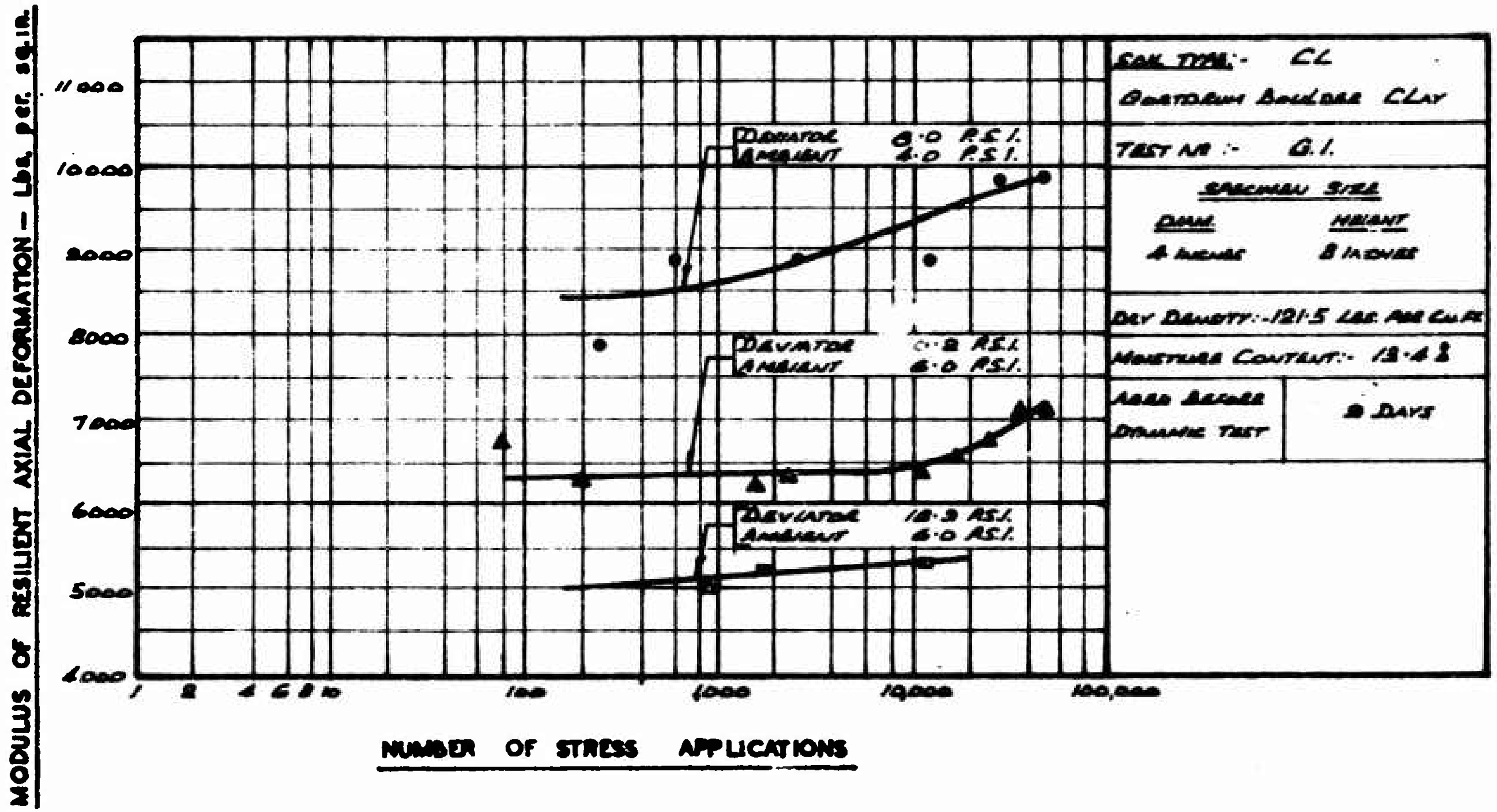
TRIAXIAL COMPRESSION TEST

FIG. 44



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

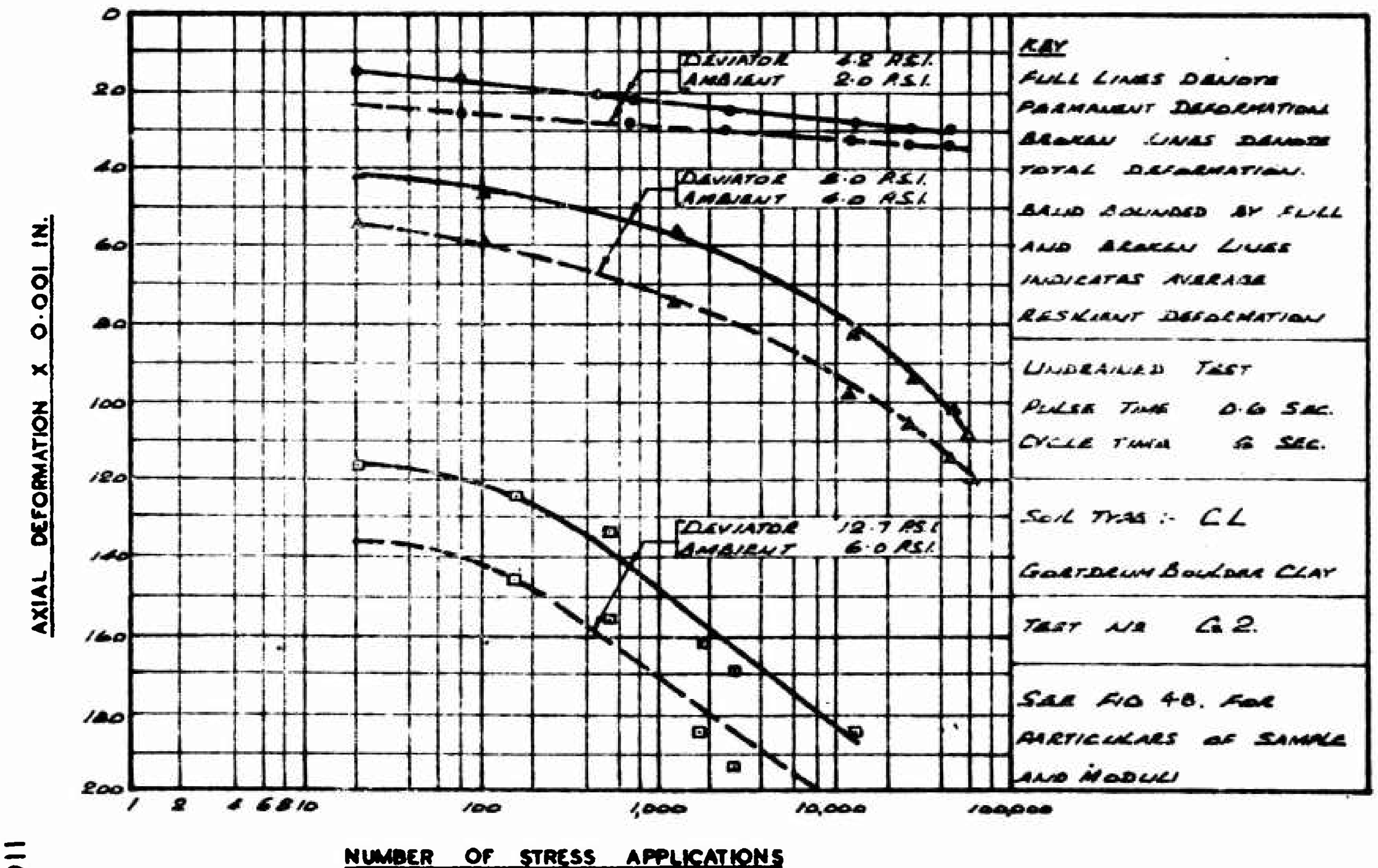
FIG. 45



DYNAMIC MODULUS – VS – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

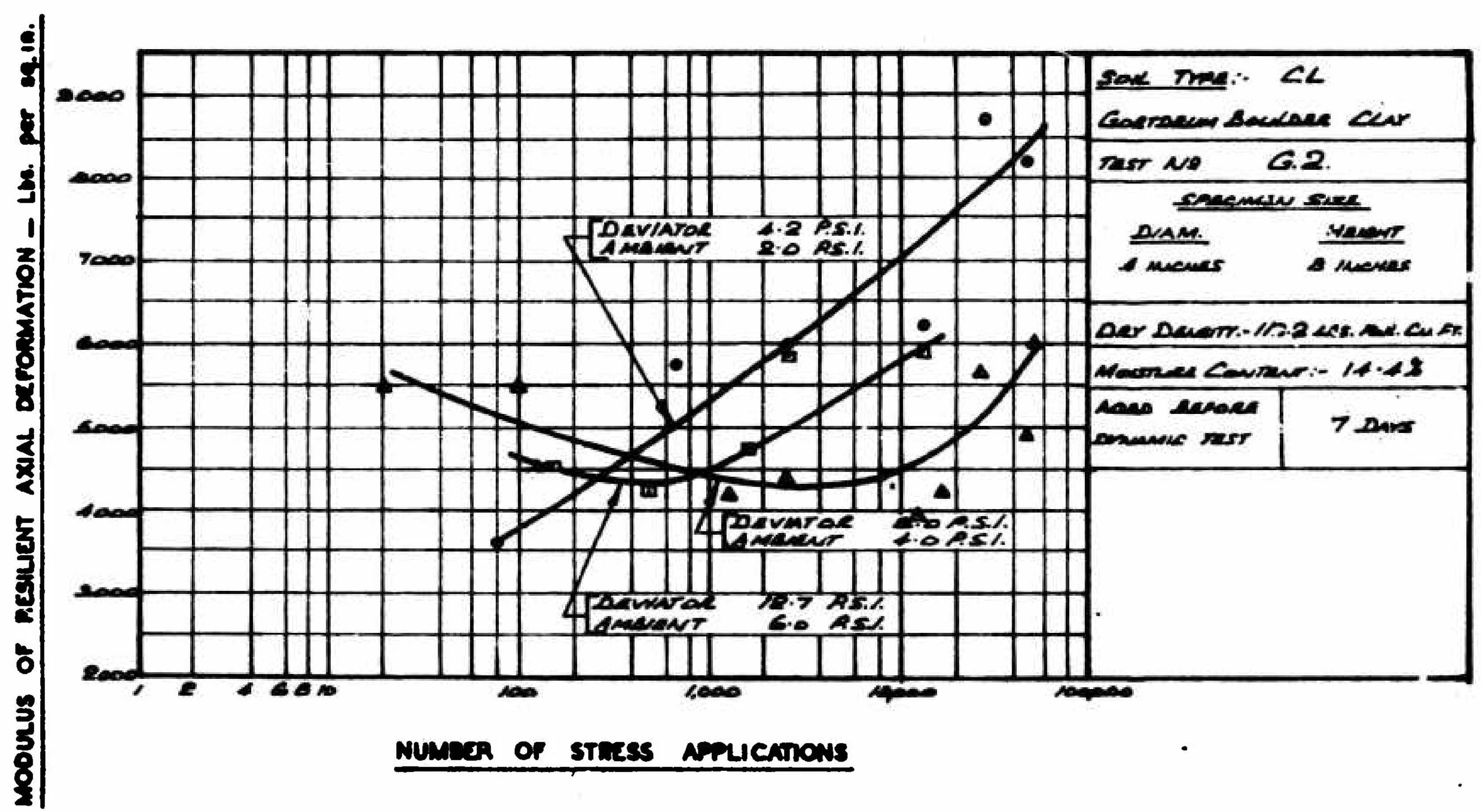
TRIAXIAL COMPRESSION TEST

FIG. 46



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

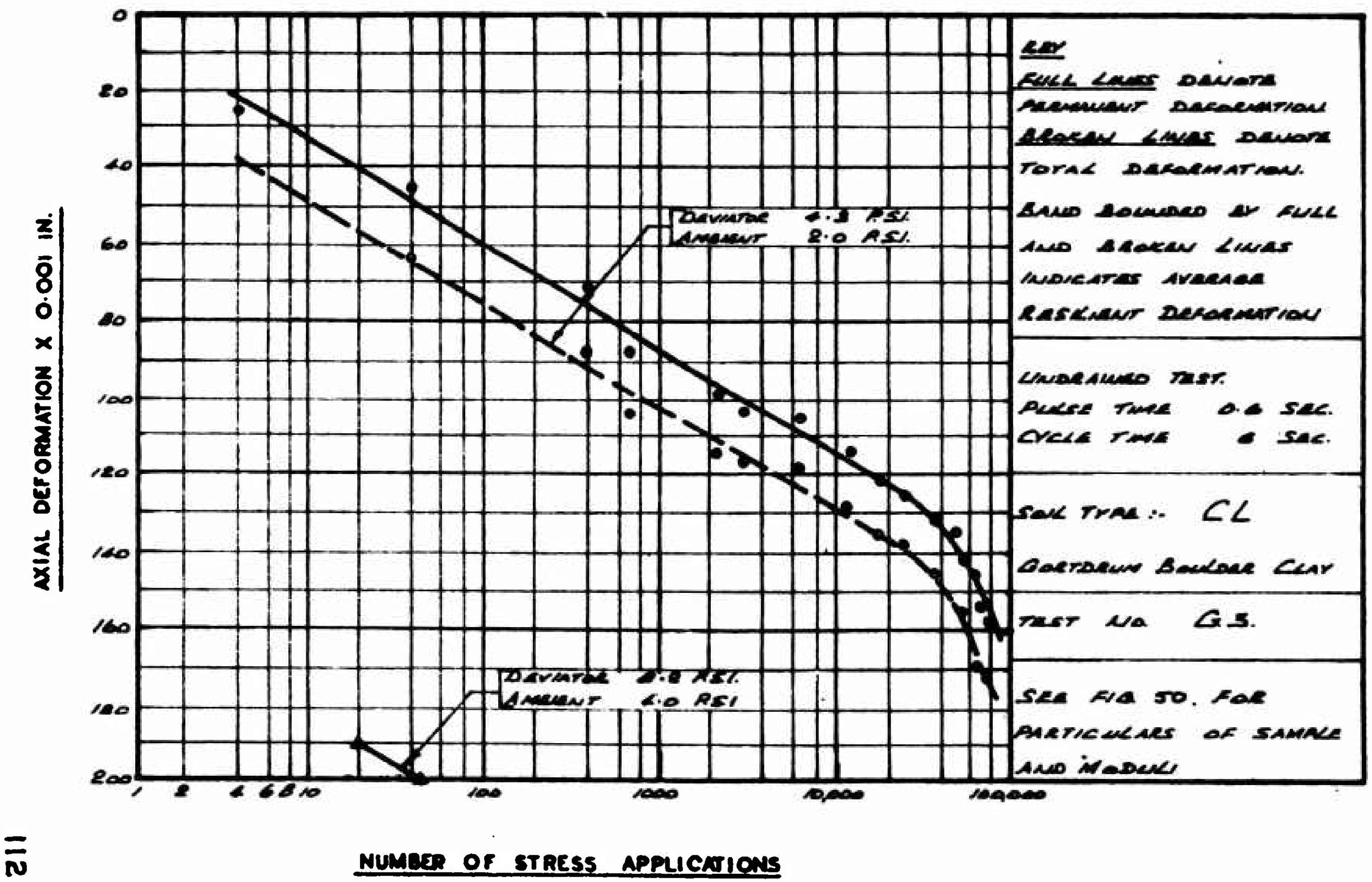
FIG. 47



DYNAMIC MODULUS — VS. — NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

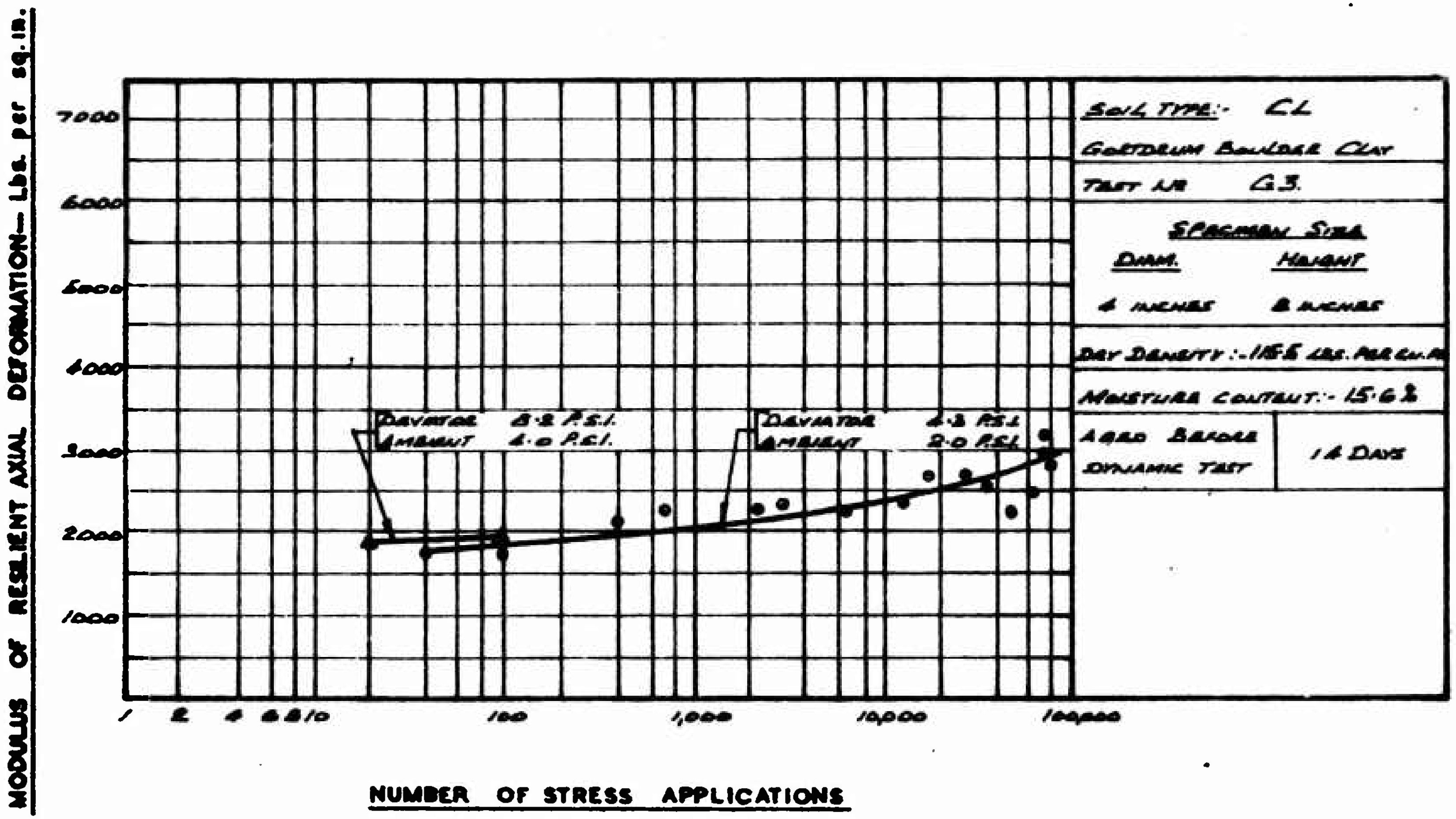
TRIAXIAL COMPRESSION TEST

FIG. 48



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

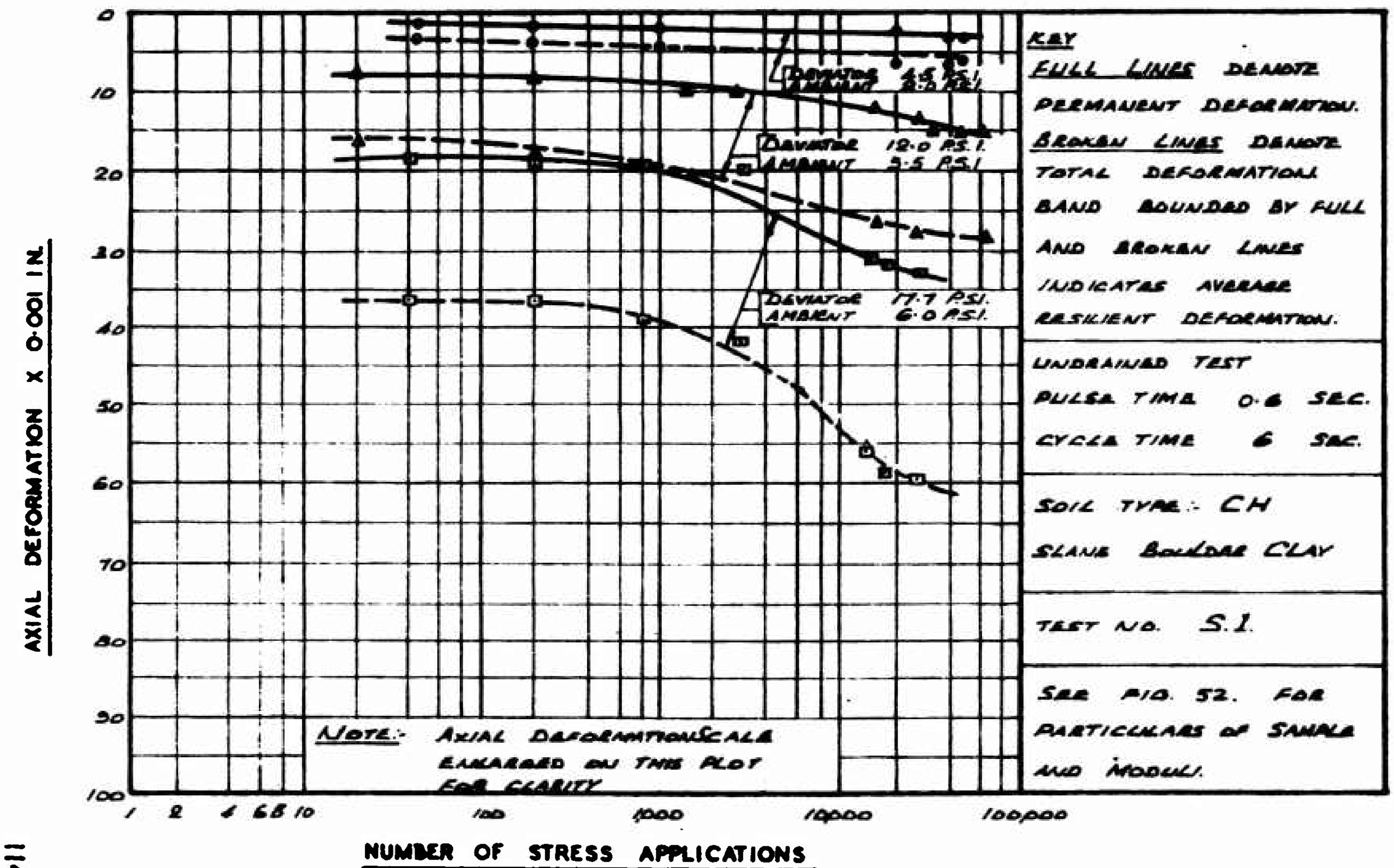
FIG. 49



DYNAMIC MODULUS – VS. – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

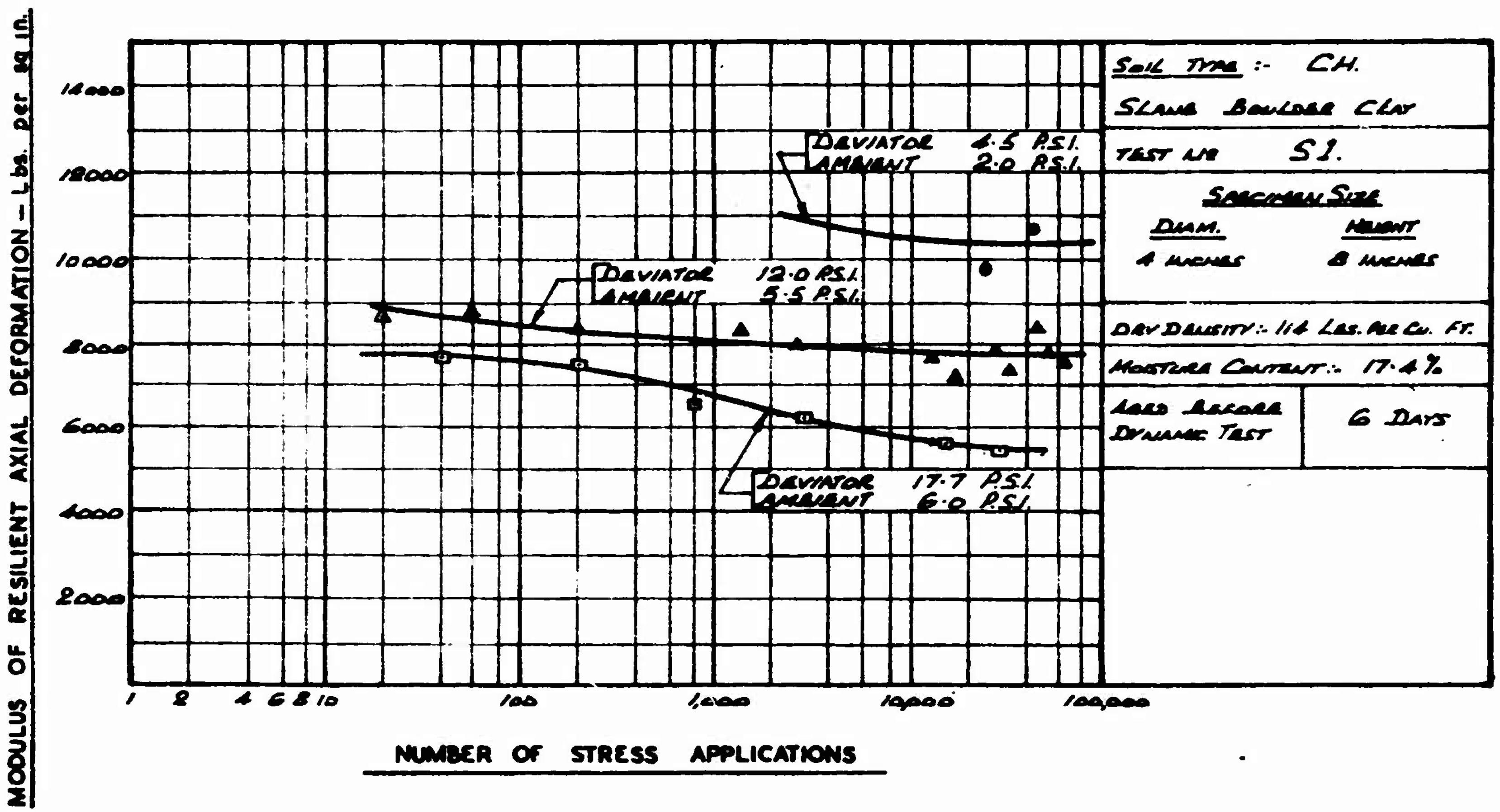
TRIAXIAL COMPRESSION TEST

FIG. 50



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

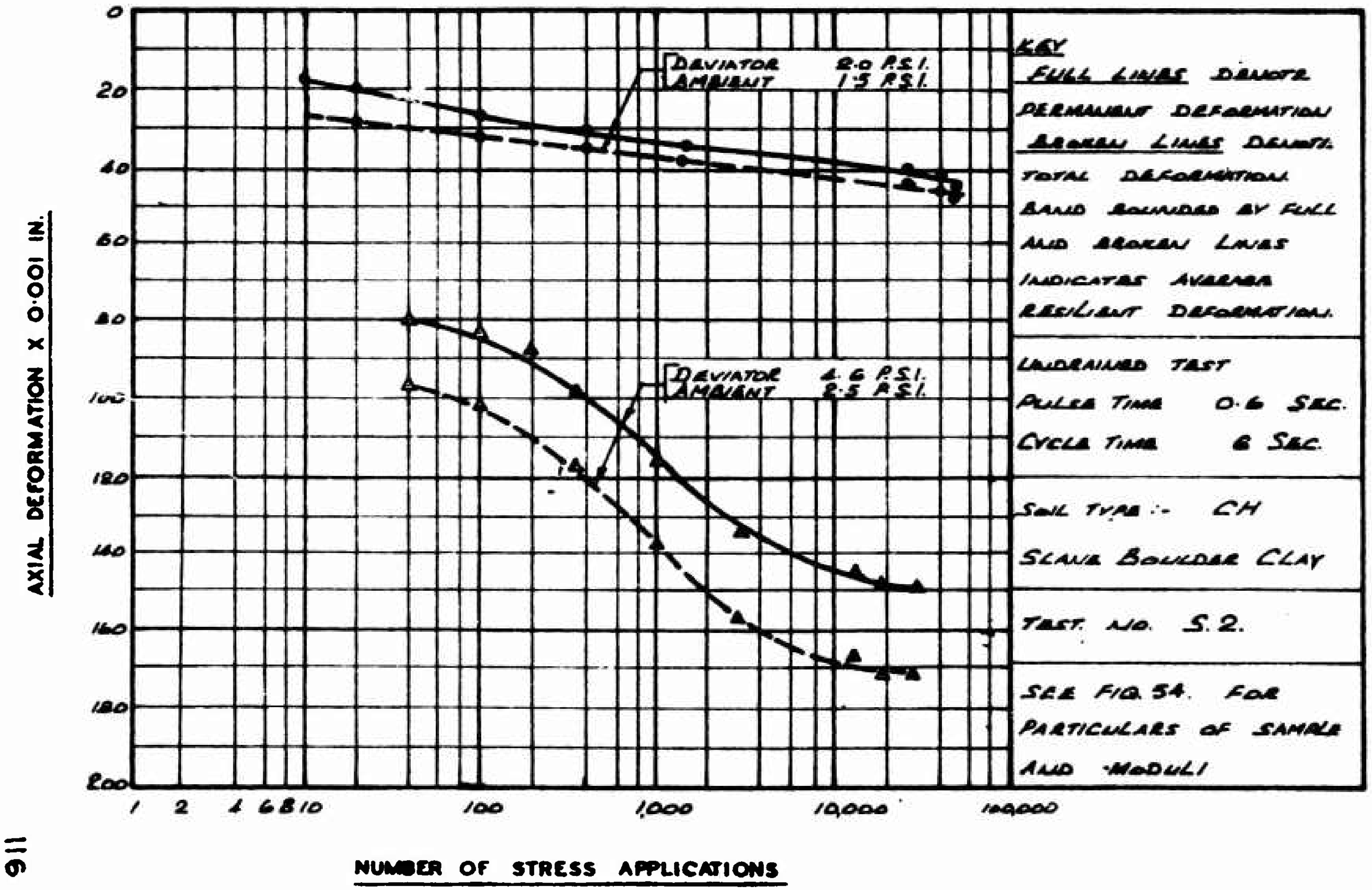
FIG. 51



DYNAMIC MODULUS — VS.— NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

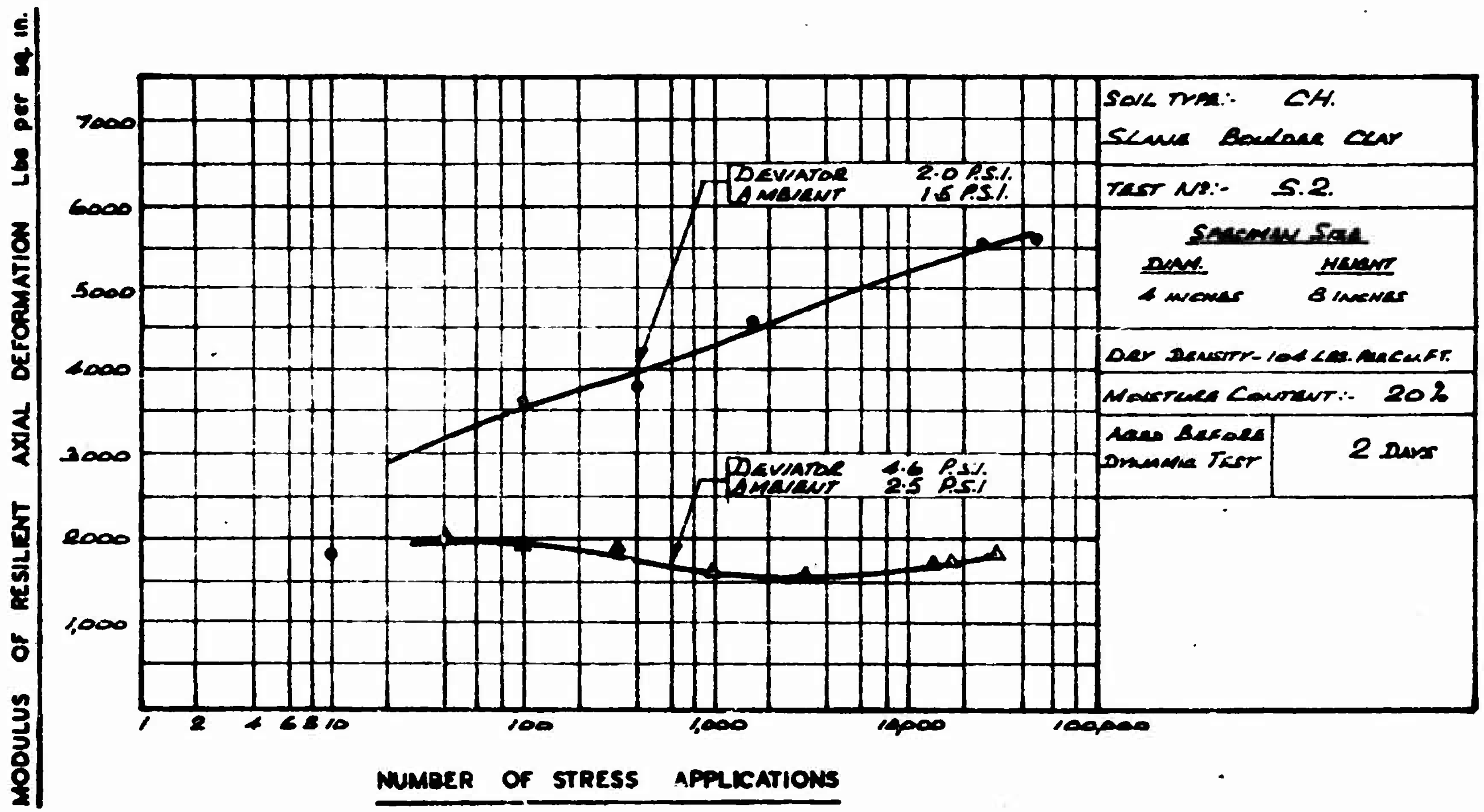
TRIAXIAL COMPRESSION TEST

FIG. 52



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

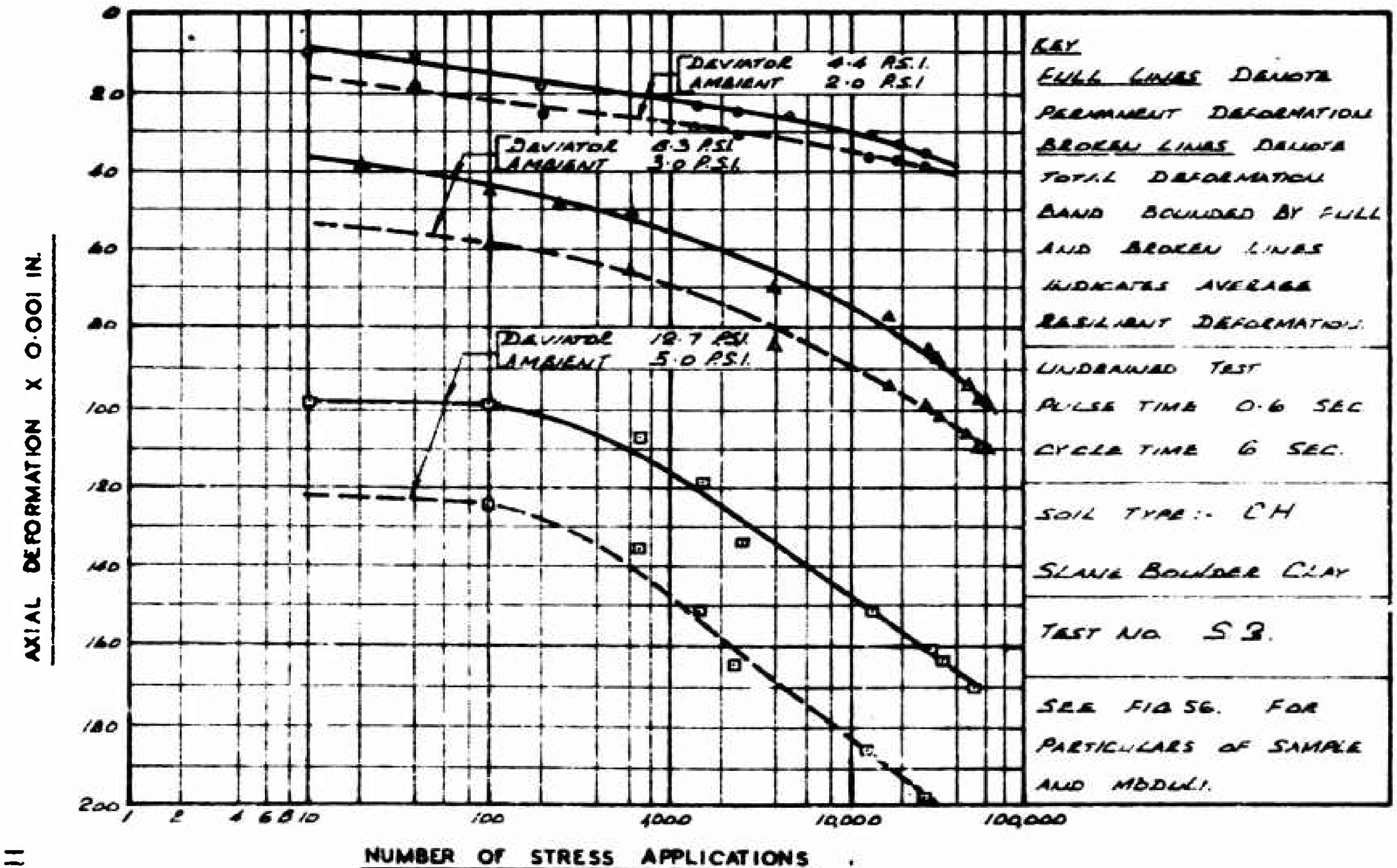
FIG. 53



DYNAMIC MODULUS – VS. – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

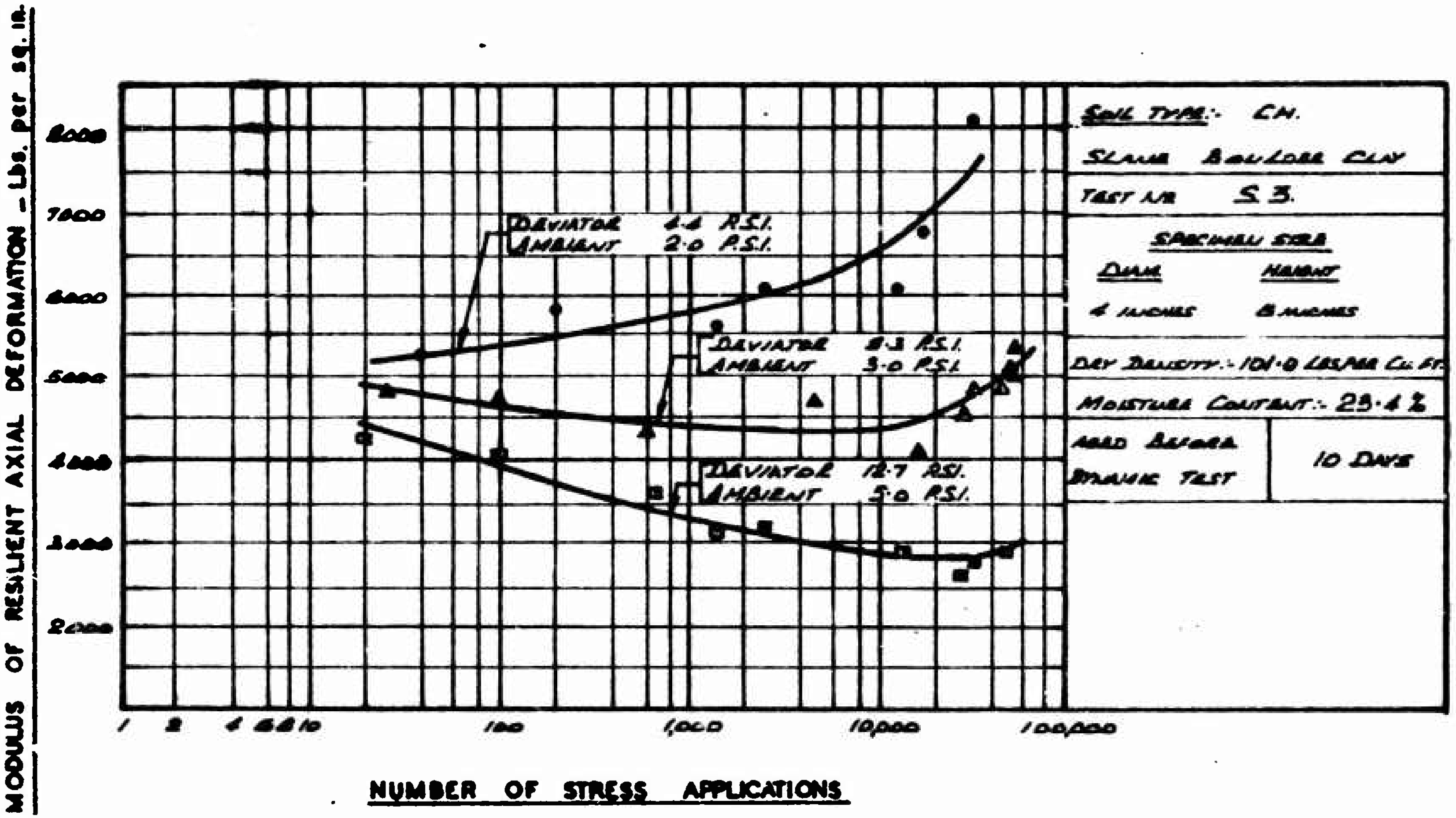
TRIAXIAL COMPRESSION TEST

FIG. 54



RESULTS OF REPEATED LOADING TRIAXIAL COMPRESSION TEST

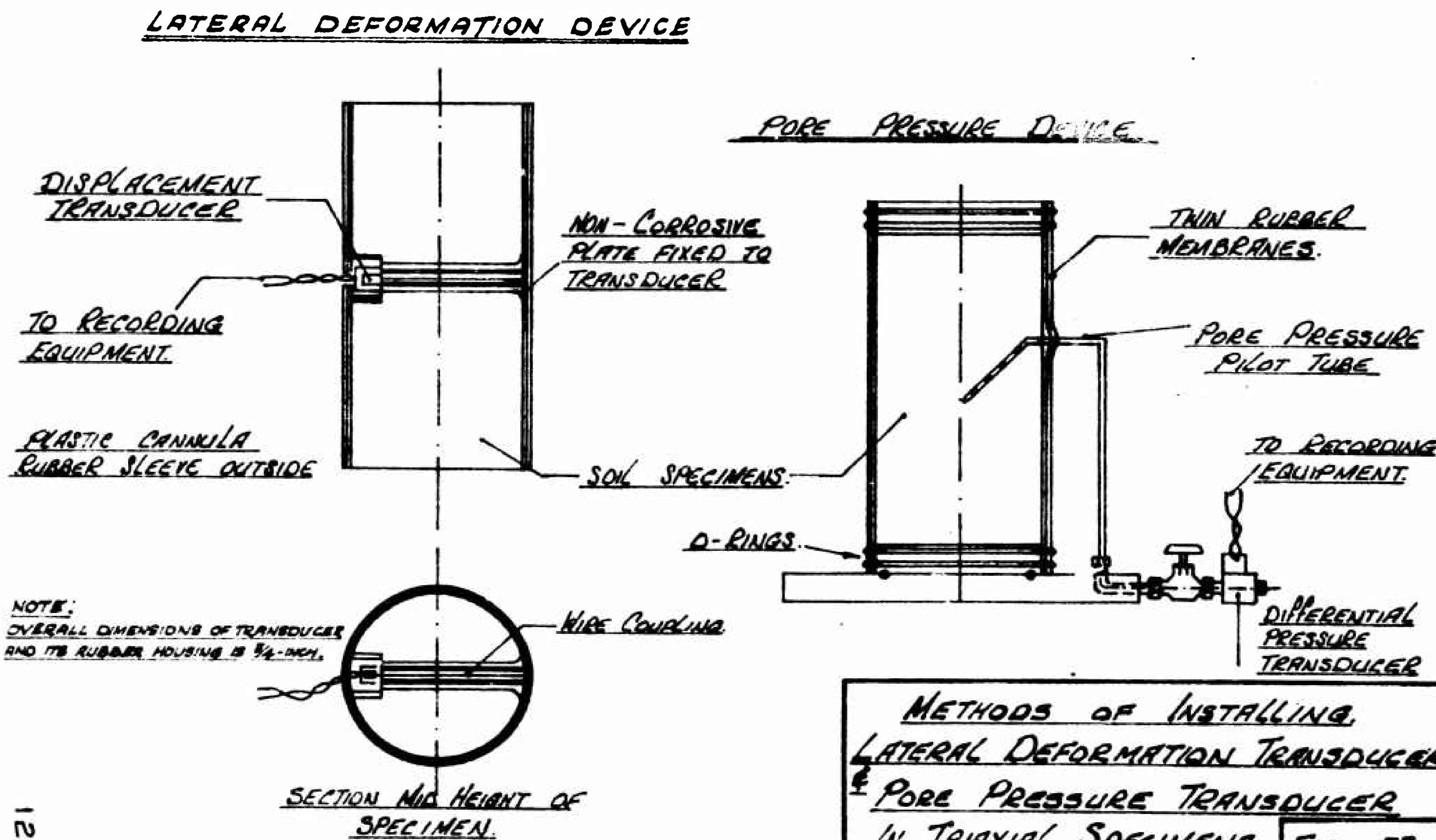
FIG. 55



DYNAMIC MODULUS – VS. – NUMBER OF STRESS APPLICATIONS IN REPEATED LOADING

TRIAXIAL COMPRESSION TEST

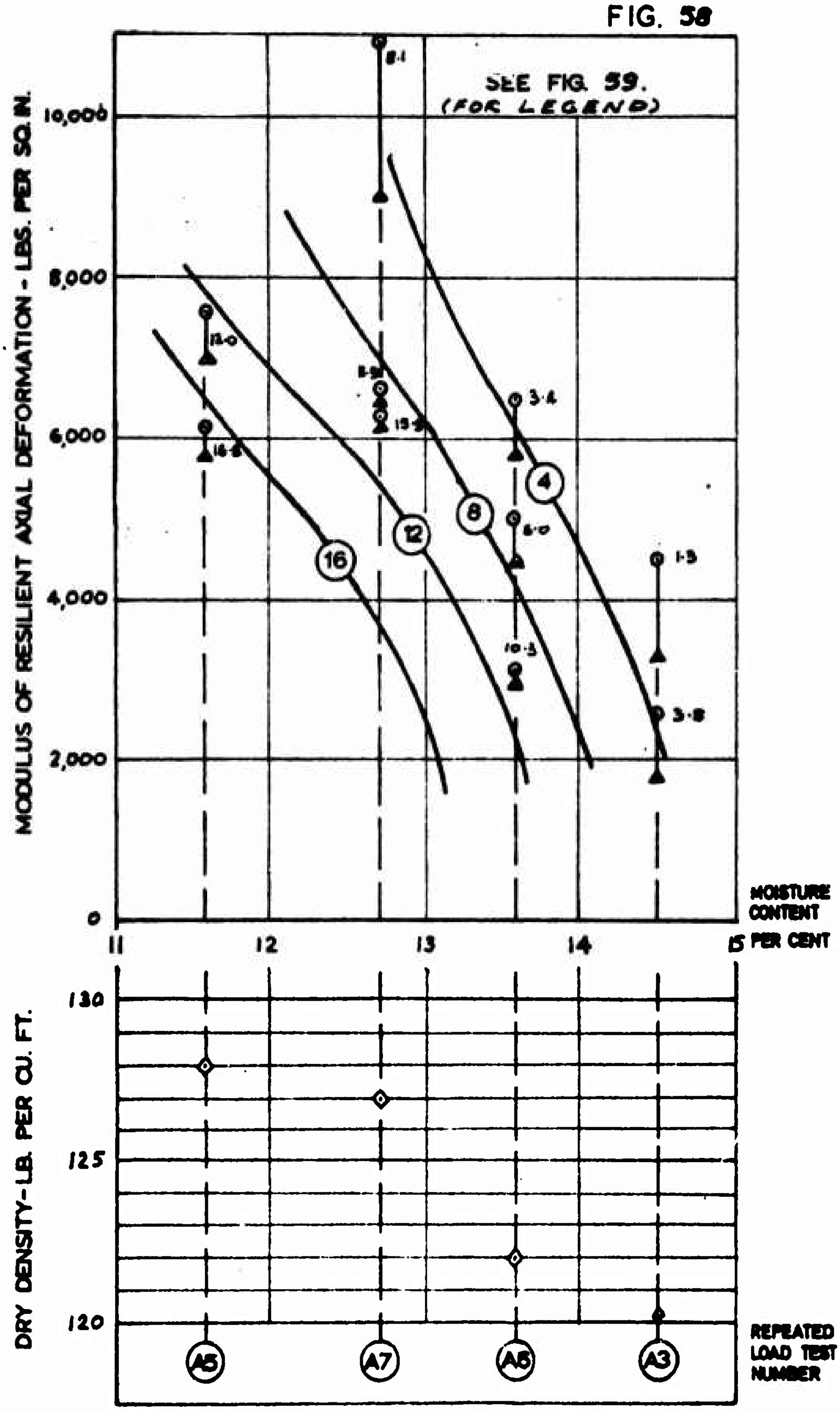
FIG. 56



METHODS OF INSTALLING LATERAL DEFORMATION TRANSDUCER & PORE PRESSURE TRANSDUCER IN TRIAXIAL SPECIMENS.

FIG. 57

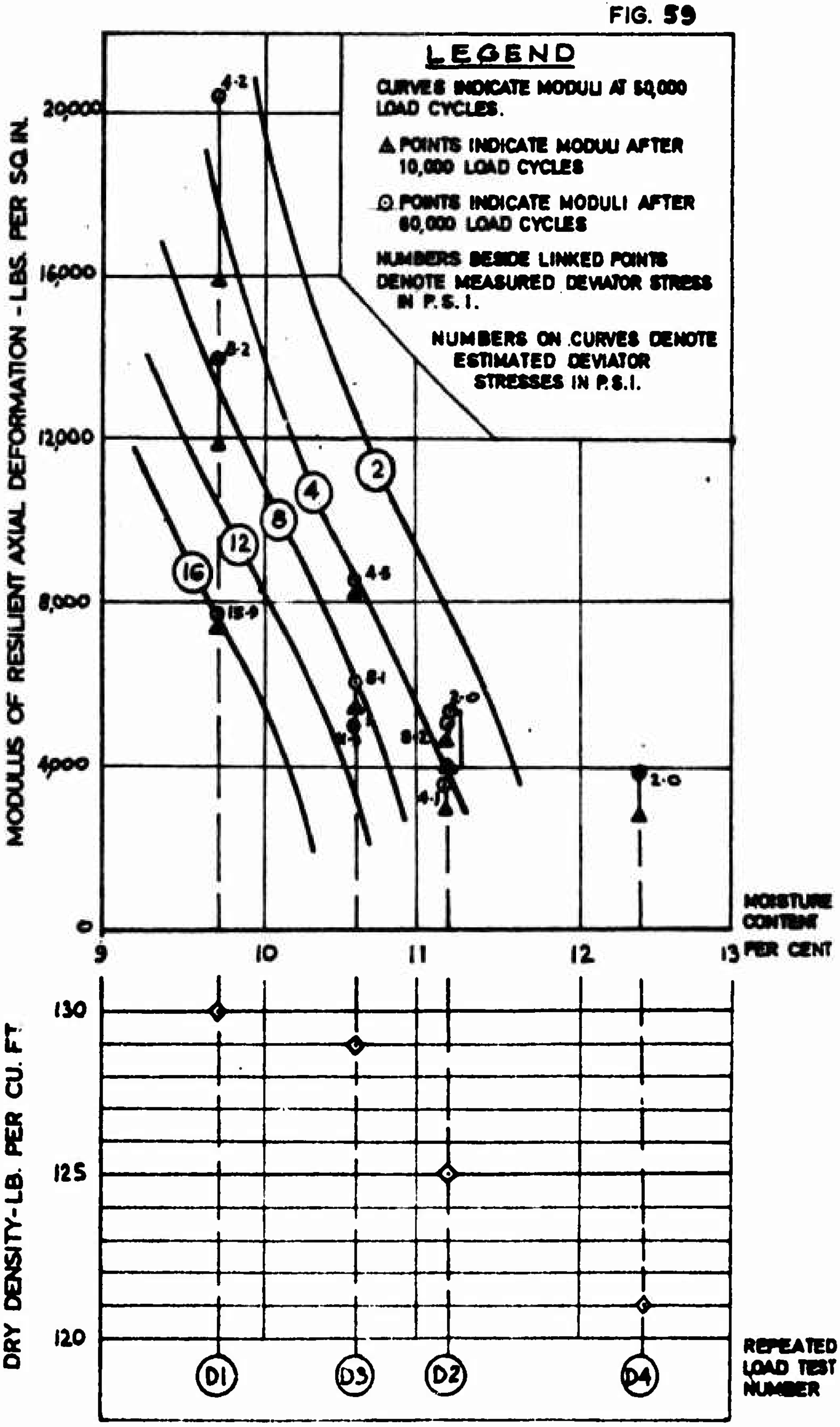
FIG. 58



RELATIONSHIP BETWEEN RESILIENT MODULUS,
DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY
IN REPEATED LOAD TESTS

ARIGNA BOULDER CLAY

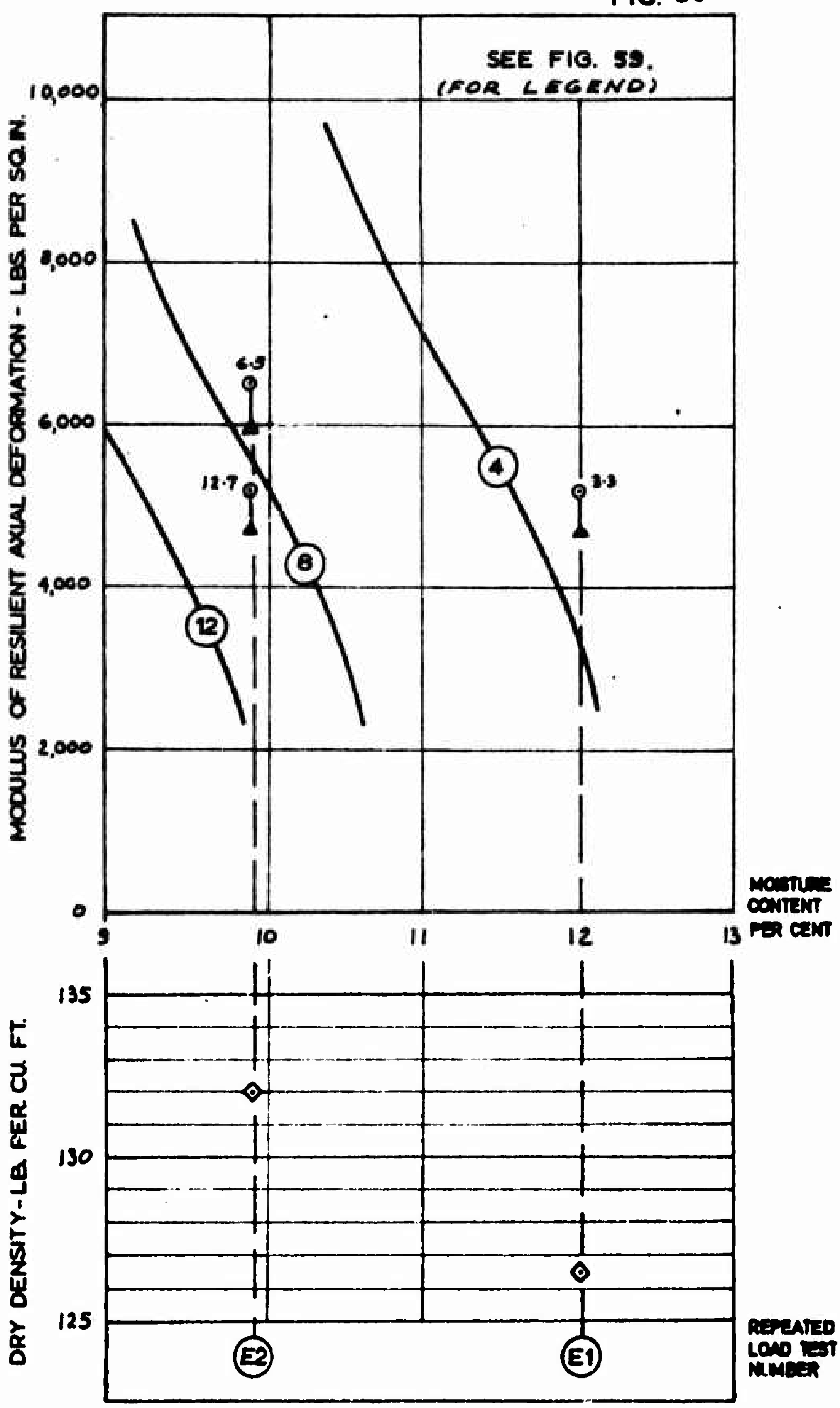
FIG. 59



RELATIONSHIP BETWEEN RESILIENT MODULUS, DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

DUBLIN BOULDER CLAY

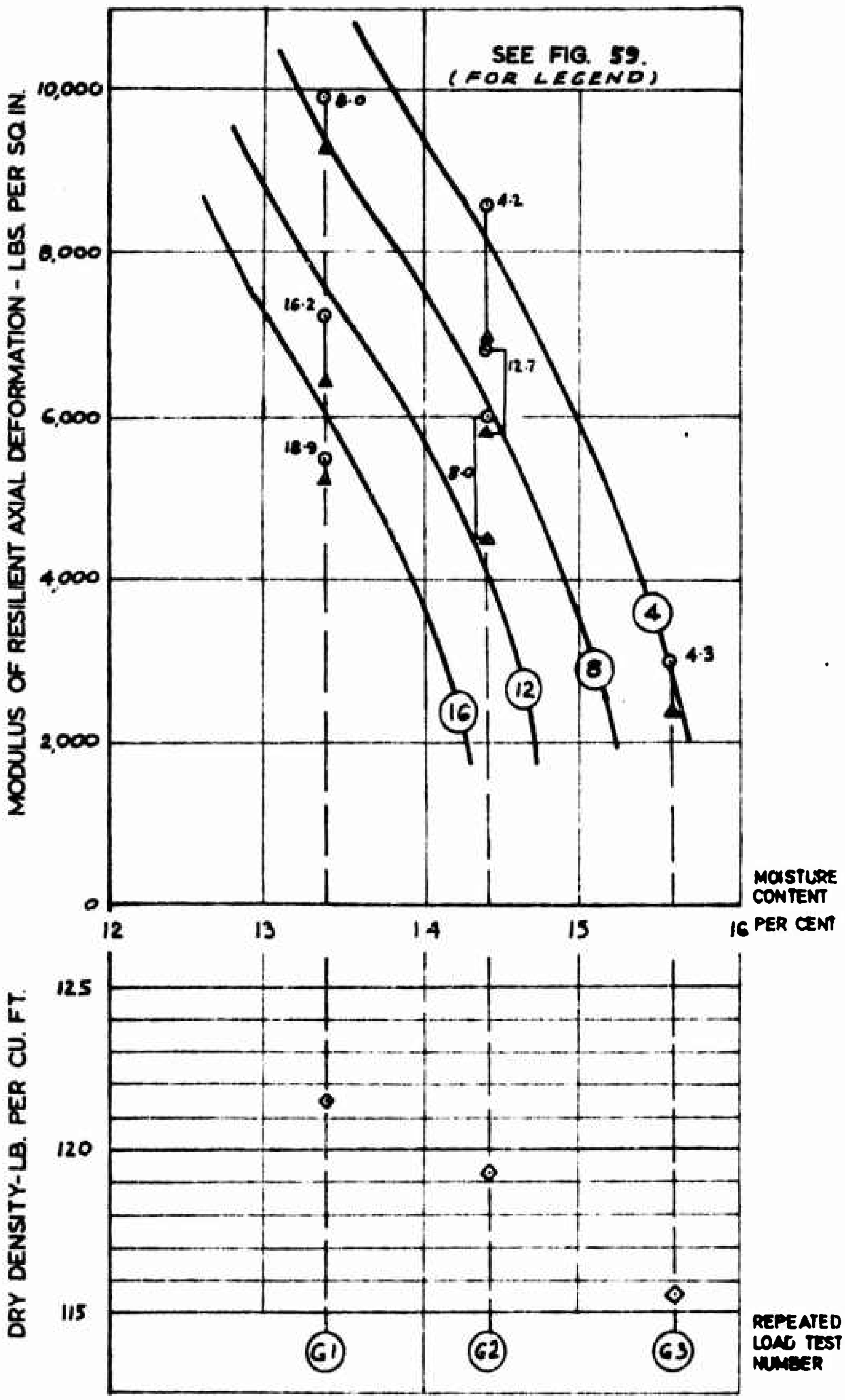
FIG. 60



RELATIONSHIP BETWEEN RESILIENT MODULUS, DEVIATOR STRESS MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

ERNE BOULDER CLAY

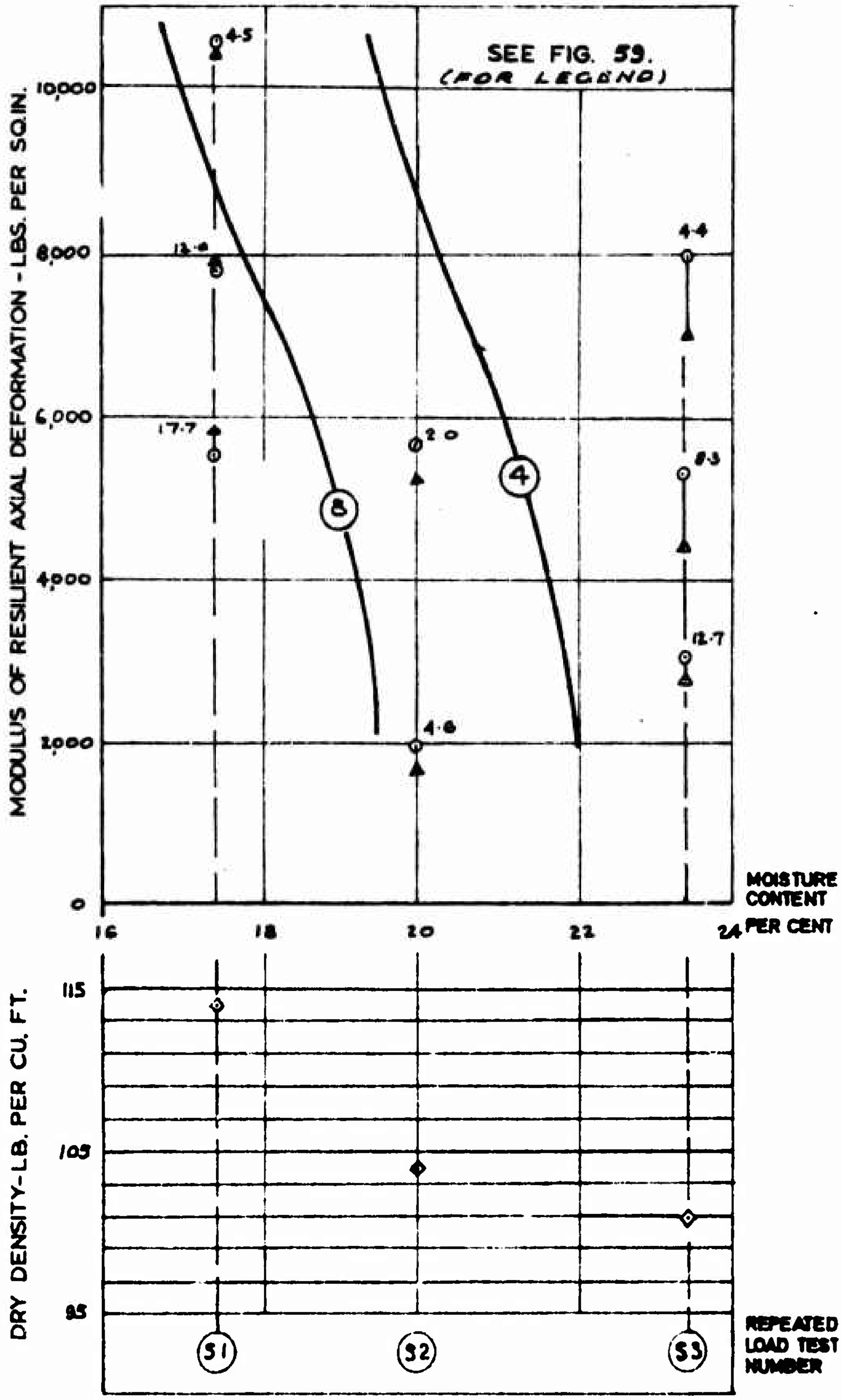
FIG. 61



RELATIONSHIP BETWEEN RESILIENT MODULUS,
DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY
IN REPEATED LOAD TESTS

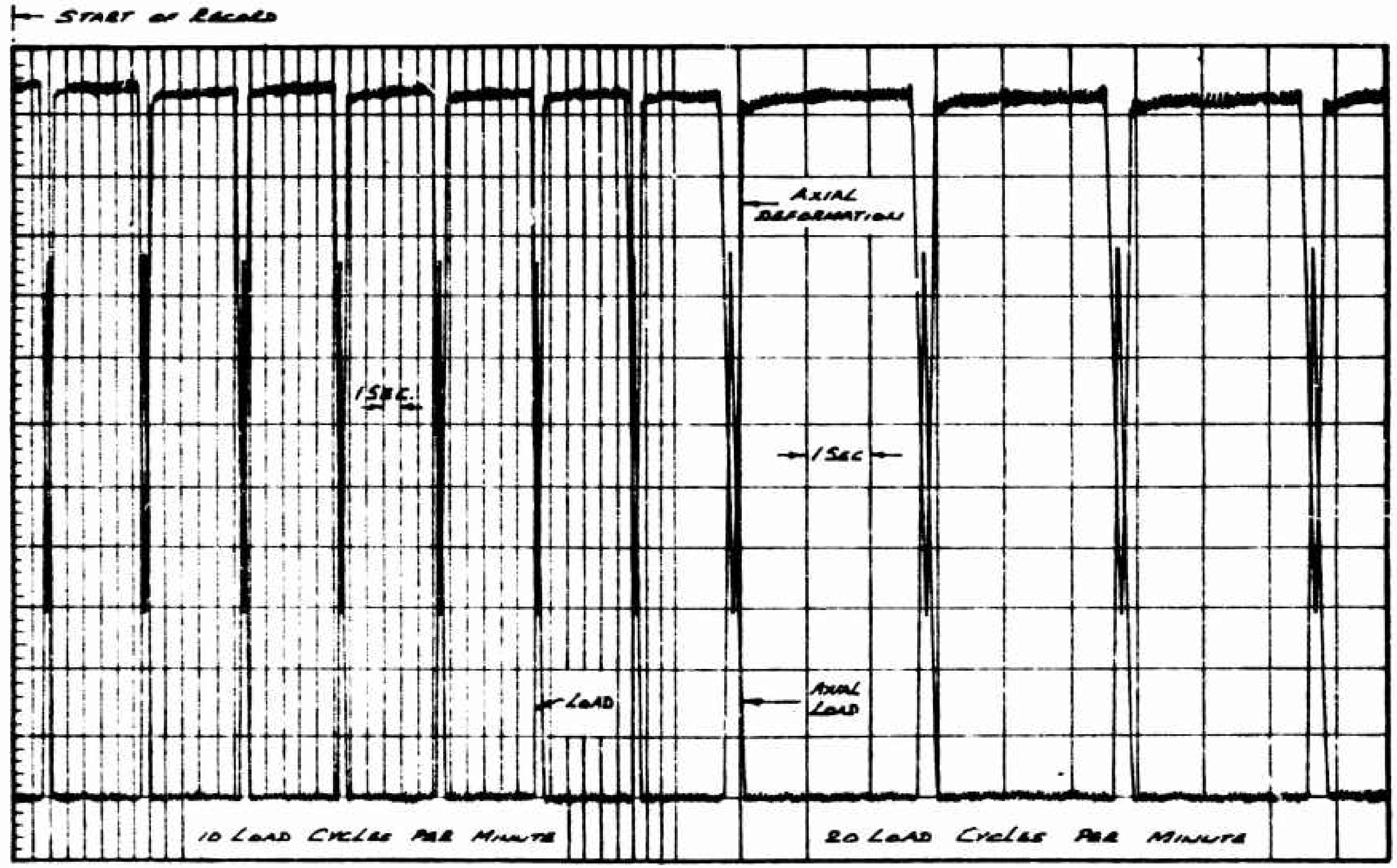
GORTDRUM BOULDER CLAY

FIG. 62



RELATIONSHIP BETWEEN RESILIENT MODULUS,
DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY
IN REPEATED LOAD TESTS

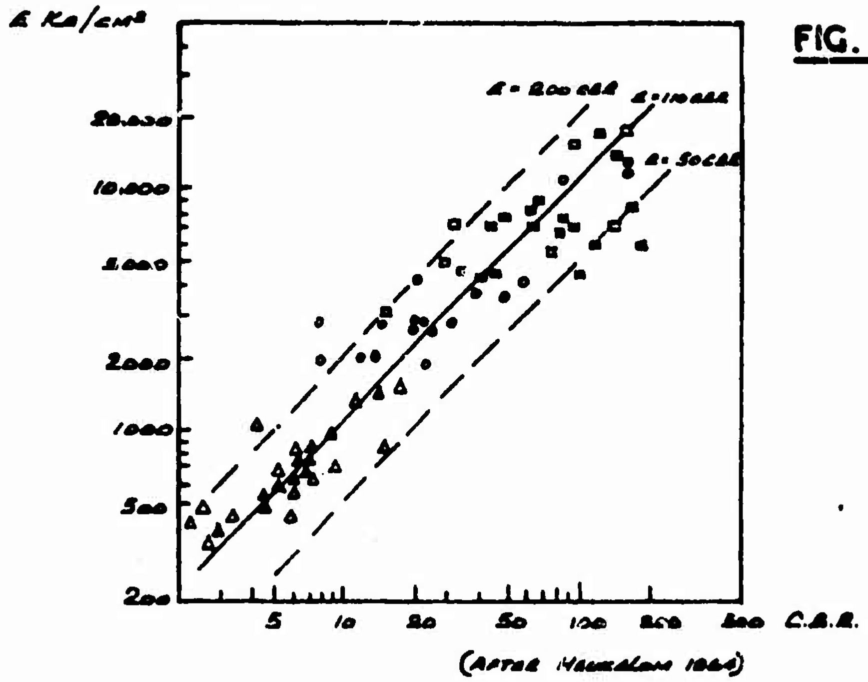
SLANE BOULDER CLAY

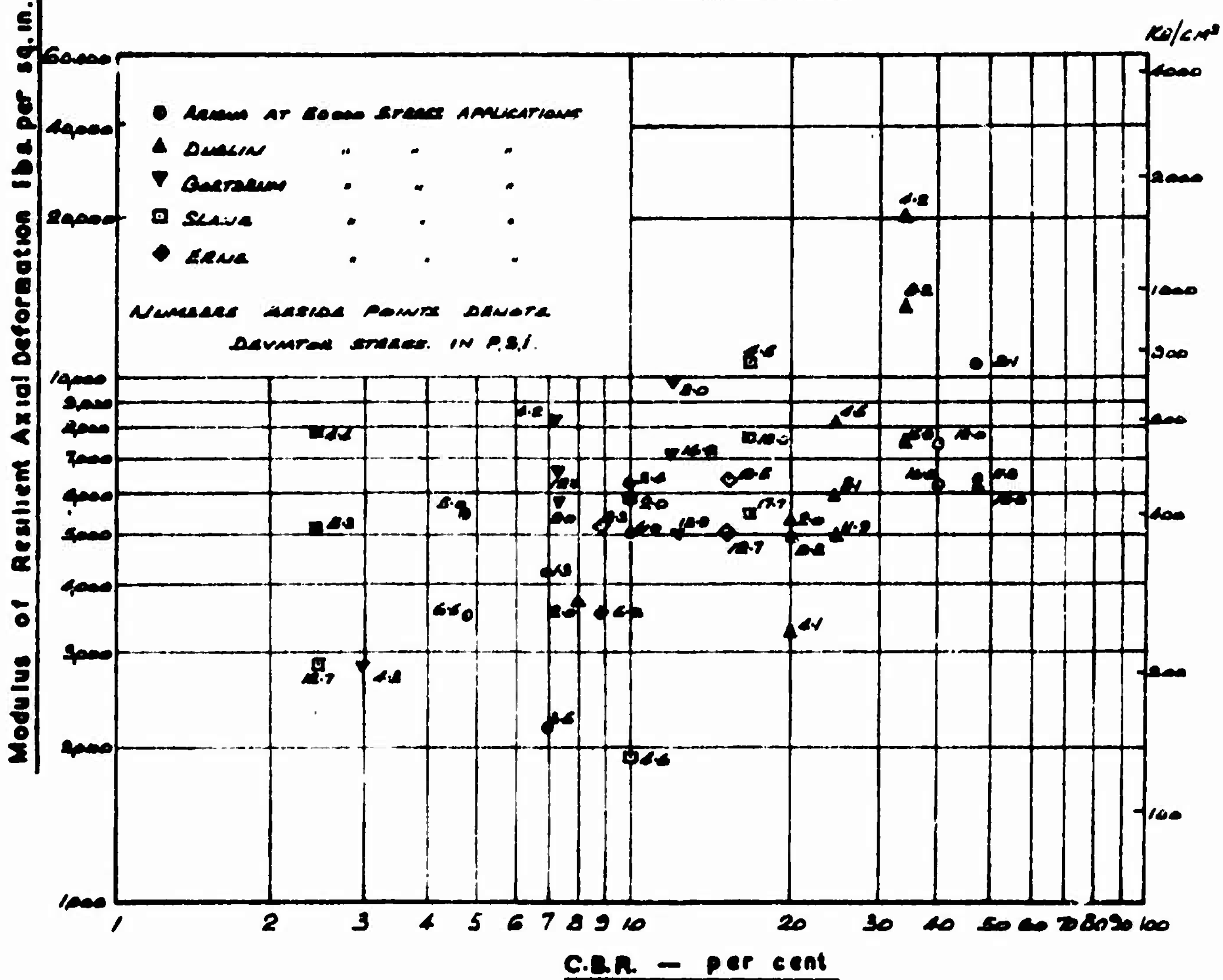


TRACES OF AXIAL LOAD AND DEFORMATION FOR DIFFERENT LOADING RATES ON SAME SAMPLE OF COMPACTED CLAY

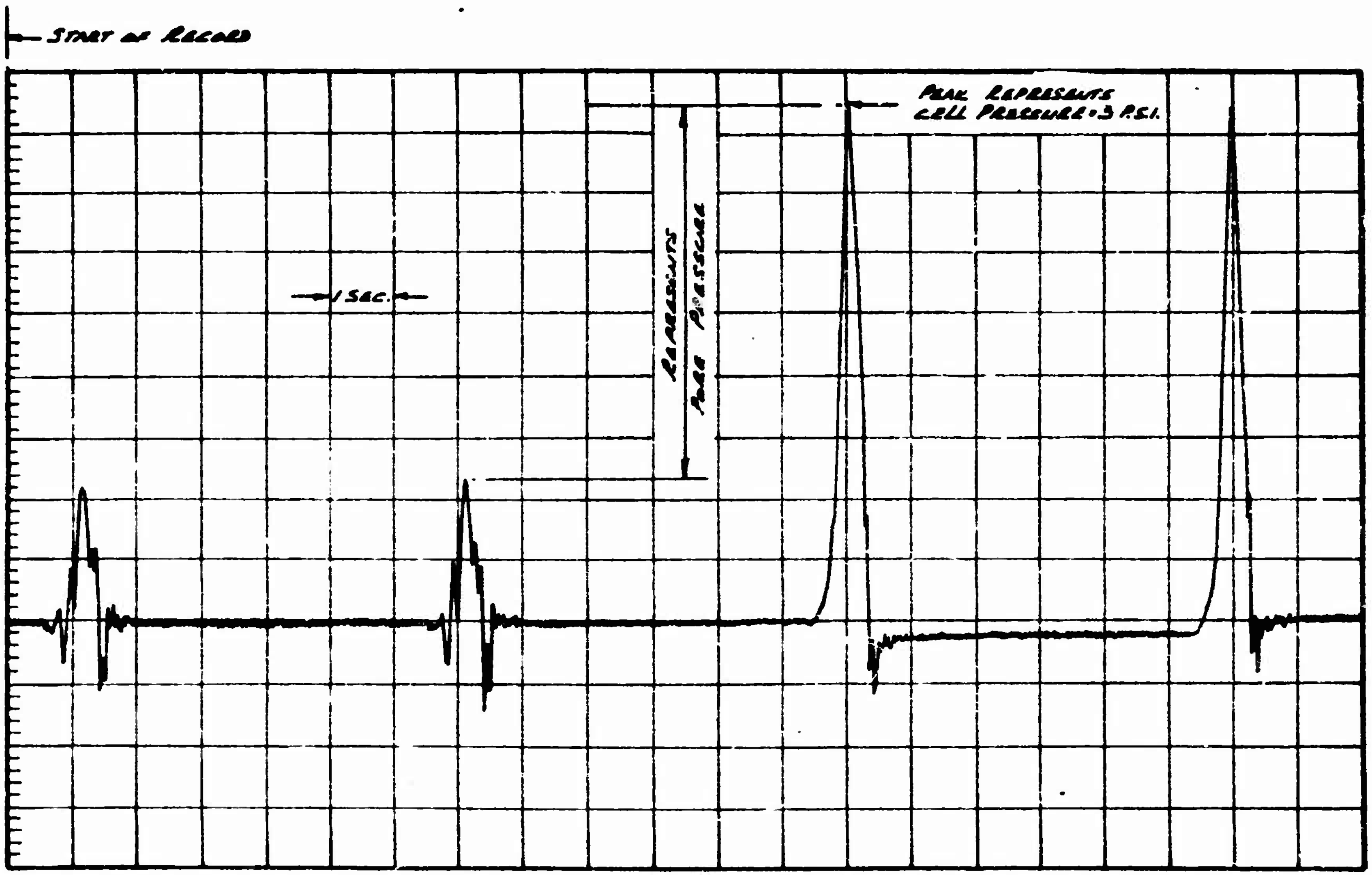
FIG. 63

FIG. 64





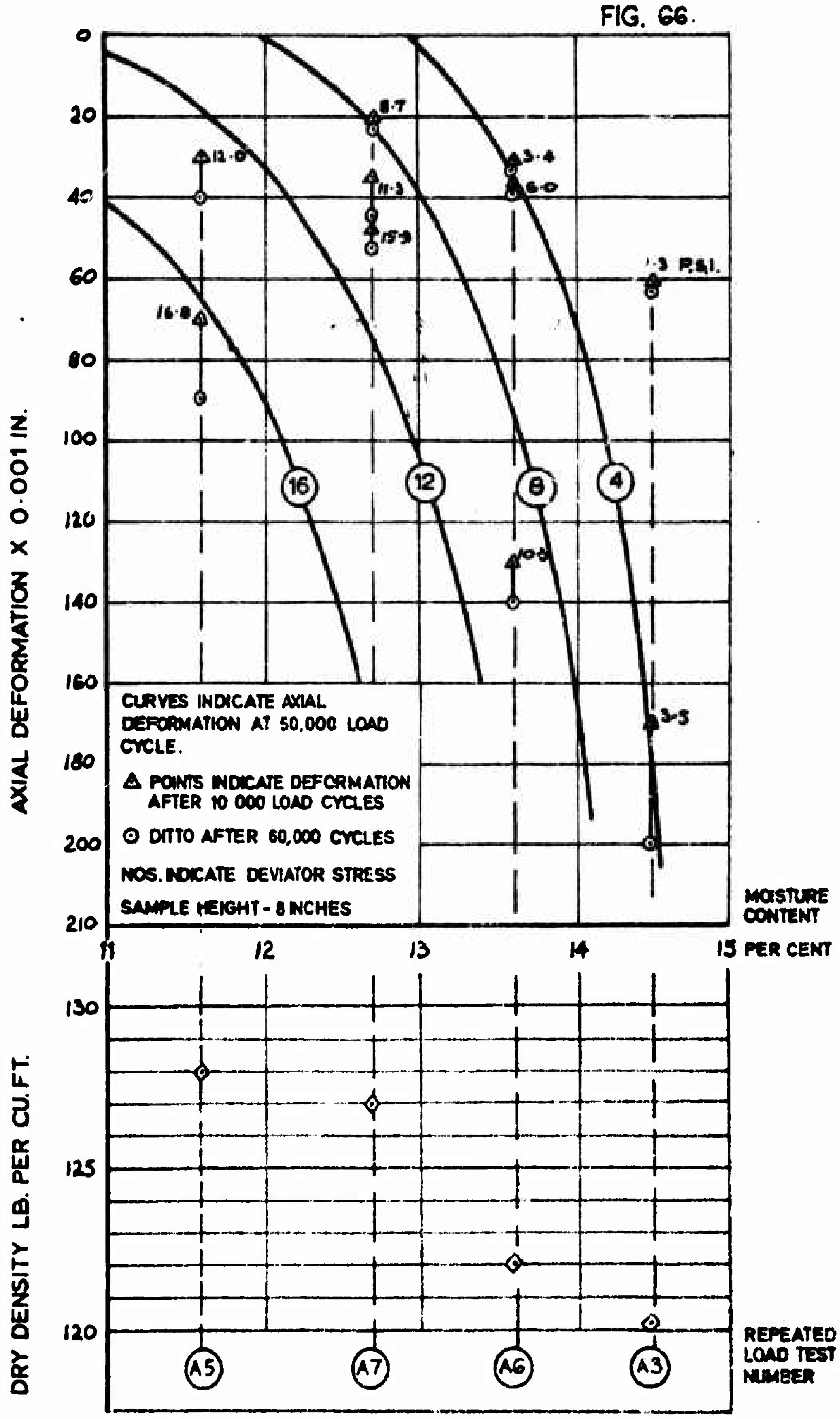
RELATIONSHIP BETWEEN DYNAMIC MODULUS AND C.B.R. VALUES FOR FIVE BOULDER CLAYS.



TRACES OBTAINED IN CELL AND PORE PRESSURE MEASUREMENTS

FIG. 65.

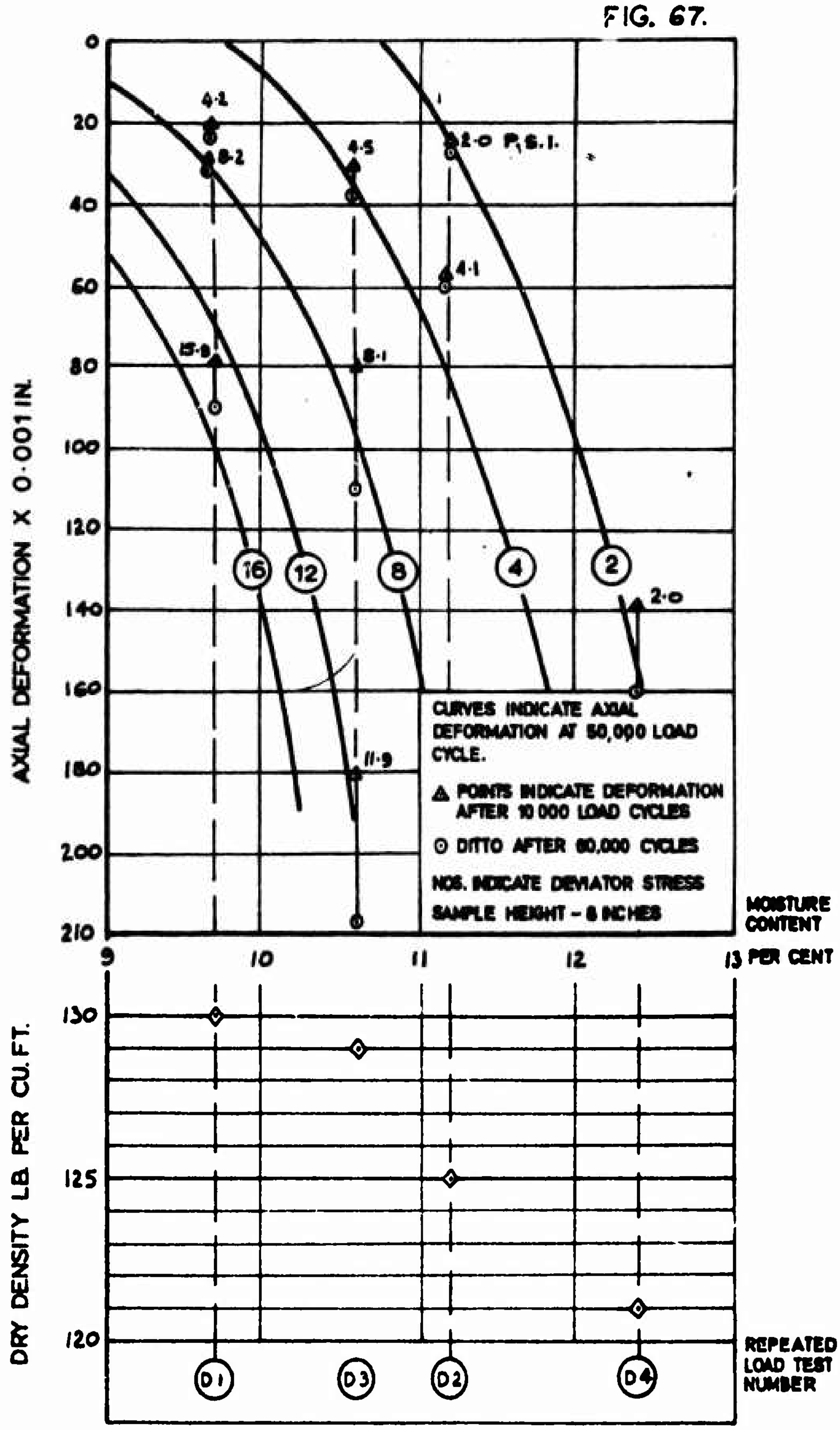
FIG. 66.



RELATIONSHIP BETWEEN PERMANENT DEFORMATION, DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

ARIGNA BOULDER CLAY

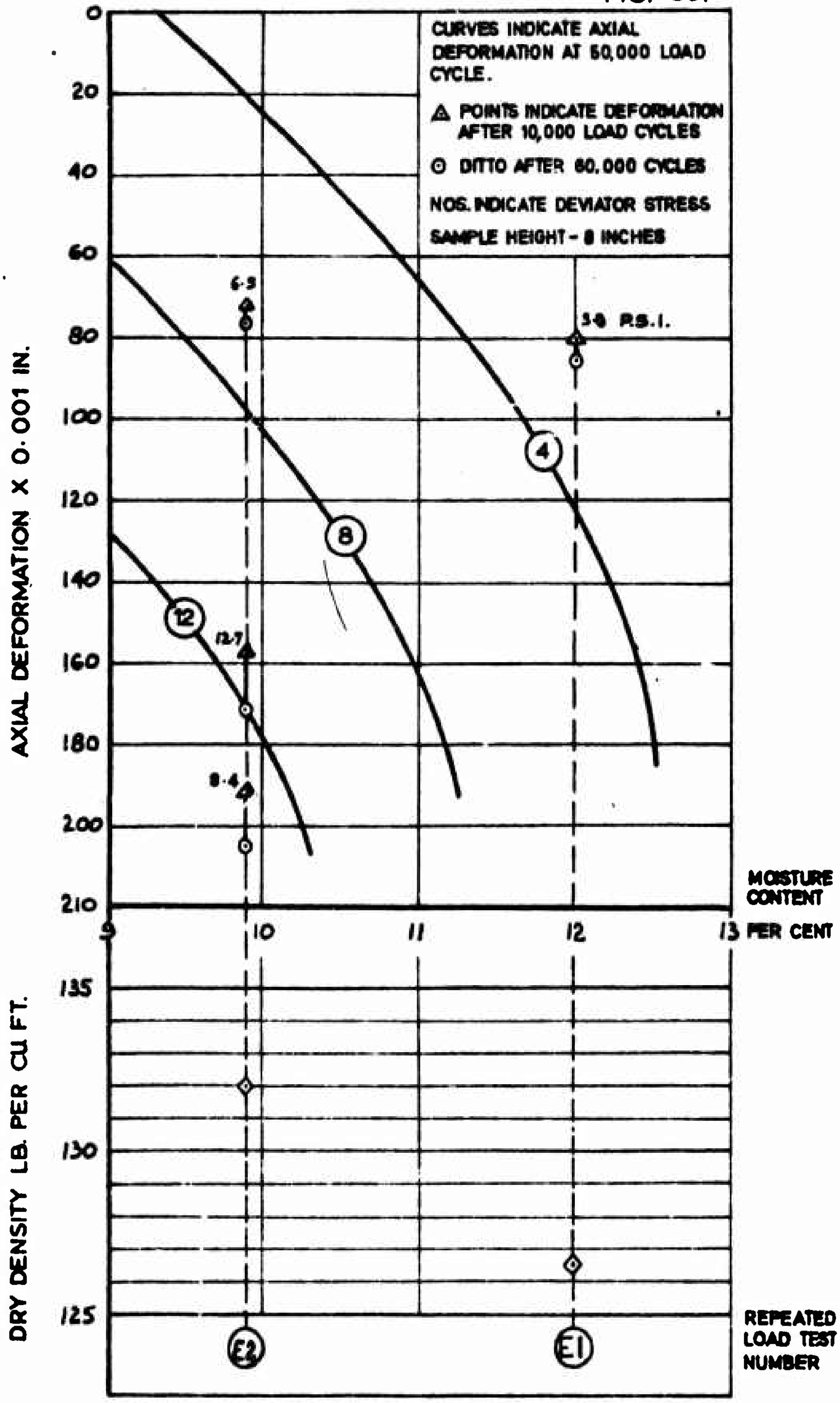
FIG. 67.



RELATIONSHIP BETWEEN PERMANENT DEFORMATION, DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

DUBLIN BOULDER CLAY

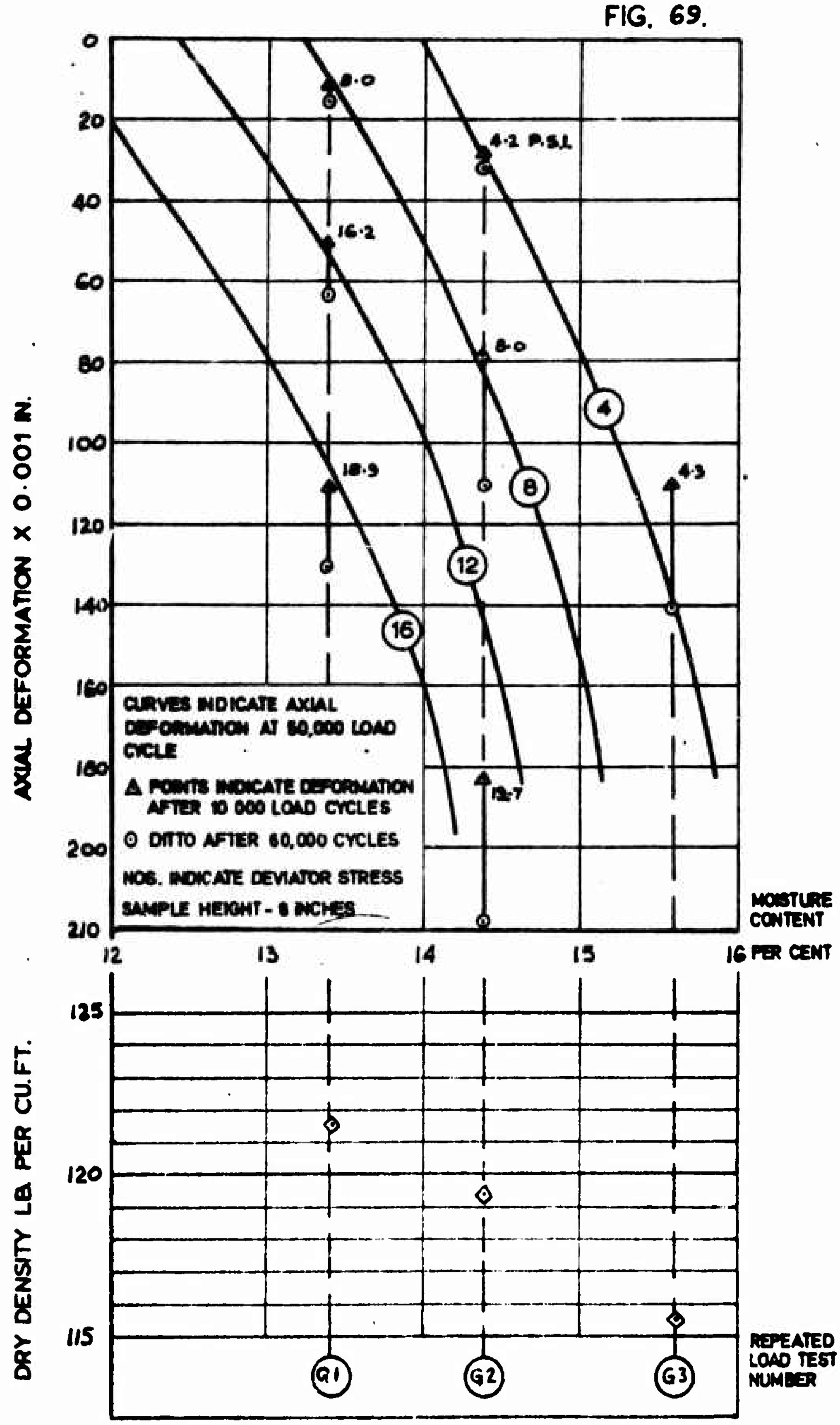
FIG. 68.



RELATIONSHIP BETWEEN PERMANENT DEFORMATION, DEVIATOR STRESS MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

ERNE BOULDER CLAY

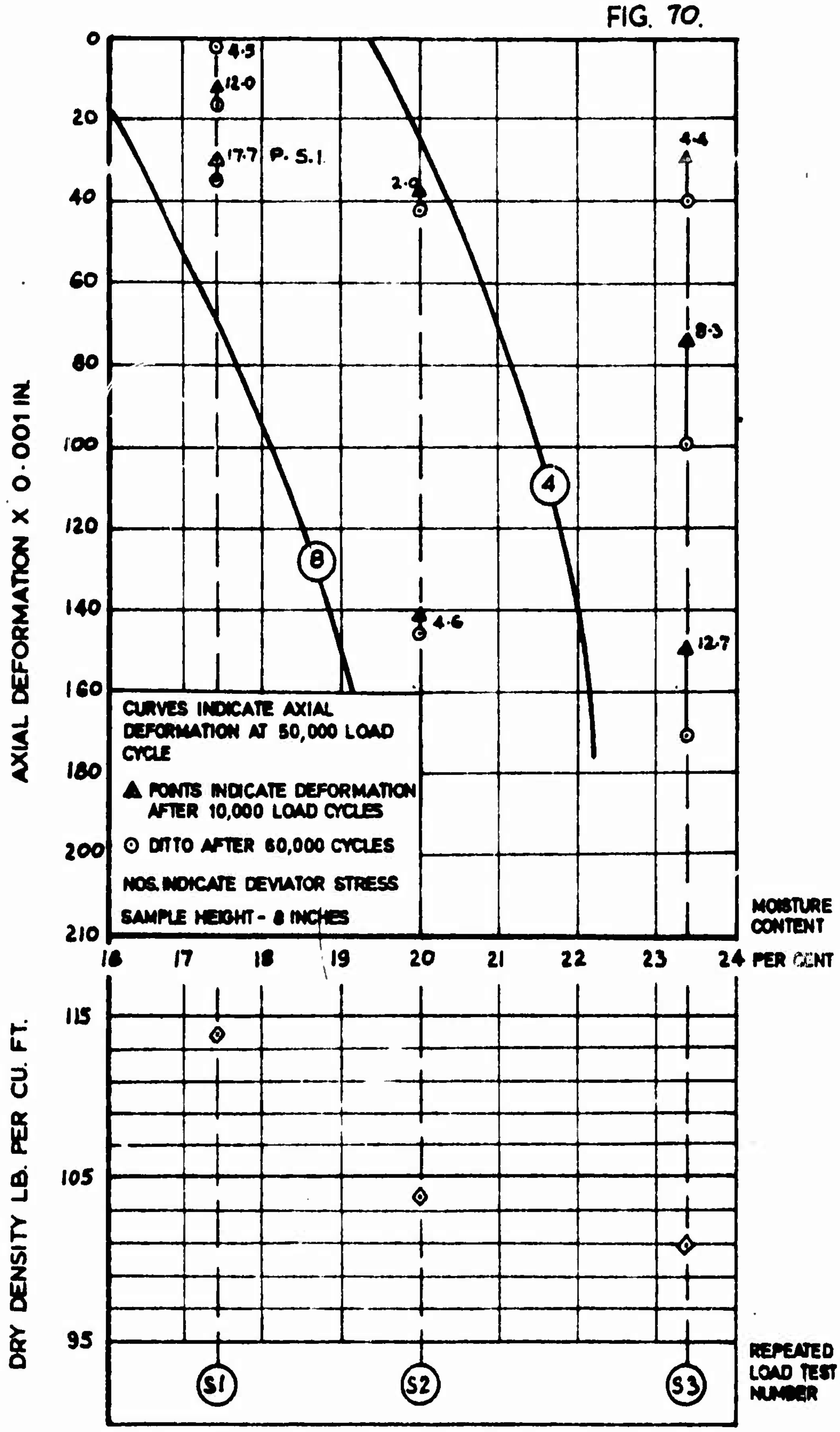
FIG. 69.



RELATIONSHIP BETWEEN PERMANENT DEFORMATION, DEVIATOR STRESS, MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

GORTDRUM BOULDER CLAY

FIG. 70.



RELATIONSHIP BETWEEN PERMANENT DEFORMATION, DEVIATOR STRESS MOISTURE CONTENT & DRY DENSITY IN REPEATED LOAD TESTS

SLANE BOULDER CLAY

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	2b. GROUP

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13. ABSTRACT

Five soils, with liquid limits in the range 25 to 75, were subjected to pulsed deviator stresses applied in increments until a limiting permanent deformation was reached, or alternatively an axial stress of 18 p.s.i. was sustained without failure within 100,000 stress applications. Resilient moduli, covering a practical range of moisture contents and densities, were determined for each soil. The threshold stress, which separates the regimes of mainly elastic from the inelastic behaviour was established by an analytical procedure. The application of the threshold stress concept to flexible pavement performance is discussed. The compaction characteristics and the C.B.R. vs moisture content and density relationships were determined, and were correlated where practicable with the results of the repeated load tests.

Conventional shear tests, slow undrained triaxial and unconfined compression, were performed. These test results are reported in abbreviated form.

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14. KEY WORDS	LINK A ROLE	LINK A WT	LINK B ROLE	LINK B WT	LINK C ROLE	LINK C WT
Soil mechanics Modulus of elasticity of clay Triaxial compression						

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